2012 IBC®
CODE AND COMMENTARY
Volume 2
PREFACE

The principal purpose of the Commentary is to provide a basic volume of knowledge and facts relating to building construction as it pertains to the regulations set forth in the 2012 International Building Code. The person who is serious about effectively designing, constructing and regulating buildings and structures will find the Commentary to be a reliable data source and reference to almost all components of the built environment.

As a follow-up to the International Building Code, we offer a companion document, the International Building Code Commentary—Volume II. Volume II covers Chapters 16 through 35 and the appendices of the 2012 International Building Code. The basic appeal of the Commentary is thus: it provides in a small package and at reasonable cost thorough coverage of many issues likely to be dealt with when using the International Building Code — and then supplements that coverage with historical and technical background. Reference lists, information sources and bibliographies are also included.

Throughout all of this, effort has been made to keep the vast quantity of material accessible and its method of presentation useful. With a comprehensive yet concise summary of each section, the Commentary provides a convenient reference for regulations applicable to the construction of buildings and structures. In the chapters that follow, discussions focus on the full meaning and implications of the code text. Guidelines suggest the most effective method of application, and the consequences of not adhering to the code text. Illustrations are provided to aid understanding; they do not necessarily illustrate the only methods of achieving code compliance.

The format of the Commentary includes the full text of each section, table and figure in the code, followed immediately by the commentary applicable to that text. At the time of printing, the Commentary reflects the most up-to-date text of the 2012 International Building Code. As stated in the preface to the International Building Code, the content of sections in the code which begin with a letter designation (i.e., Section [P]2903.1) are maintained by another code development committee. Each section's narrative includes a statement of its objective and intent, and usually includes a discussion about why the requirement commands the conditions set forth. Code text and commentary text are easily distinguished from each other. All code text is shown as it appears in the International Building Code, and all commentary is indented below the code text and begins with the symbol •.

Readers should note that the Commentary is to be used in conjunction with the International Building Code and not as a substitute for the code. The Commentary is advisory only; the code official alone possesses the authority and responsibility for interpreting the code.

Comments and recommendations are encouraged, for through your input, we can improve future editions. Please direct your comments to the Codes and Standards Development Department at the Chicago District Office.

The International Code Council would like to extend its thanks to the following individuals for their contributions to the technical content of this commentary:

Chris Marlon Gregory Cahanin
Jeff Tubbs David Cooper
Rebecca Quinn Dave Collins
Joann Surmar Vickie Lovell
James Milke John Valluits
Richard Walke Marcelo Hirschler
Dave Adams Edward Keith
Zeno Martin Phillip Samblanet

Jason Thompson
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>CHAPTER</th>
<th>TITLE</th>
<th>SECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>STRUCTURAL DESIGN</td>
<td>16-1 – 16-90</td>
</tr>
<tr>
<td>17</td>
<td>SPECIAL INSPECTIONS AND TESTS</td>
<td>17-1 – 17-30</td>
</tr>
<tr>
<td>18</td>
<td>SOILS AND FOUNDATIONS</td>
<td>18-1 – 18-80</td>
</tr>
<tr>
<td>19</td>
<td>CONCRETE</td>
<td>19-1 – 19-30</td>
</tr>
<tr>
<td>20</td>
<td>ALUMINUM</td>
<td>20-1 – 20-2</td>
</tr>
<tr>
<td>21</td>
<td>MASONRY</td>
<td>21-1 – 21-46</td>
</tr>
<tr>
<td>22</td>
<td>STEEL</td>
<td>22-1 – 22-10</td>
</tr>
<tr>
<td>23</td>
<td>WOOD</td>
<td>23-1 – 23-118</td>
</tr>
<tr>
<td>24</td>
<td>GLASS AND GLAZING</td>
<td>24-1 – 24-28</td>
</tr>
<tr>
<td>25</td>
<td>GYPSUM BOARD AND PLASTER</td>
<td>25-1 – 25-24</td>
</tr>
<tr>
<td>26</td>
<td>PLASTIC</td>
<td>26-1 – 26-28</td>
</tr>
<tr>
<td>27</td>
<td>ELECTRICAL</td>
<td>27-1 – 27-4</td>
</tr>
<tr>
<td>28</td>
<td>MECHANICAL SYSTEMS</td>
<td>28-1 – 28-2</td>
</tr>
<tr>
<td>29</td>
<td>PLUMBING SYSTEMS</td>
<td>29-1 – 29-18</td>
</tr>
<tr>
<td>30</td>
<td>ELEVATORS AND CONVEYING SYSTEMS</td>
<td>30-1 – 30-24</td>
</tr>
<tr>
<td>31</td>
<td>SPECIAL CONSTRUCTION</td>
<td>31-1 – 31-20</td>
</tr>
<tr>
<td>32</td>
<td>ENCROACHMENTS INTO THE PUBLIC RIGHT-OF-WAY</td>
<td>32-1 – 32-6</td>
</tr>
<tr>
<td>33</td>
<td>SAFEGUARDS DURING CONSTRUCTION</td>
<td>33-1 – 33-8</td>
</tr>
<tr>
<td>34</td>
<td>EXISTING STRUCTURES</td>
<td>34-1 – 34-58</td>
</tr>
<tr>
<td>35</td>
<td>REFERENCED STANDARDS</td>
<td>35-1 – 35-26</td>
</tr>
<tr>
<td>A</td>
<td>EMPLOYEE QUALIFICATIONS</td>
<td>A-1 – A-2</td>
</tr>
<tr>
<td>B</td>
<td>BOARD OF APPEALS</td>
<td>B-1 – B-4</td>
</tr>
<tr>
<td>C</td>
<td>GROUP U — AGRICULTURAL BUILDINGS</td>
<td>C-1 – C-4</td>
</tr>
</tbody>
</table>

2012 INTERNATIONAL BUILDING CODE® COMMENTARY
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appendix D</td>
<td>FIRE DISTRICTS</td>
<td>D-1 – D-8</td>
</tr>
<tr>
<td>Appendix E</td>
<td>SUPPLEMENTARY ACCESSIBILITY REQUIREMENTS</td>
<td>E-1 – E-12</td>
</tr>
<tr>
<td>Appendix F</td>
<td>RODENTPROOFING</td>
<td>F-1 – F-2</td>
</tr>
<tr>
<td>Appendix G</td>
<td>FLOOD-RESISTANT CONSTRUCTION</td>
<td>G-1 – G-16</td>
</tr>
<tr>
<td>Appendix H</td>
<td>SIGNS</td>
<td>H-1 – H-10</td>
</tr>
<tr>
<td>Appendix I</td>
<td>PATIO COVERS</td>
<td>I-1 – I-2</td>
</tr>
<tr>
<td>Appendix J</td>
<td>GRADING</td>
<td>J-1 – J-8</td>
</tr>
<tr>
<td>Appendix K</td>
<td>ADMINISTRATIVE PROVISIONS</td>
<td>K-1 – K-4</td>
</tr>
<tr>
<td>Appendix L</td>
<td>EARTHQUAKE RECORDING INSTRUMENTATION</td>
<td>L-1 – L-2</td>
</tr>
<tr>
<td>Appendix M</td>
<td>TSUNAMI-GENERATED FLOOD HAZARD</td>
<td>M-1 – M-4</td>
</tr>
<tr>
<td>Index</td>
<td></td>
<td>INDEX-1 – INDEX-38</td>
</tr>
</tbody>
</table>
Chapter 16: Structural Design

General Comments
This chapter contains the commentary for the following structural topics: definitions of structural terms, construction document requirements, load combinations, dead loads, live loads, snow loads, wind loads, soil lateral loads, rain loads, flood loads and earthquake loads. This chapter provides minimum design requirements so that all buildings and structures are proportioned to resist the loads and forces that are likely to be encountered. The loads specified herein have been established through research and service performance of buildings and structures. The application of these loads and adherence to the serviceability criteria will enhance the protection of life and property. The earthquake loads, wind loads and snow loads in this chapter are based on the 2010 edition of ASCE 7. The earthquake criteria and ASCE 7 load requirements are based on the National Earthquake Hazards Reduction Program’s (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and other Structures (FEMA 450). The NEHRP provisions were prepared by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA).

Purpose
The purpose of this chapter is to prescribe minimum structural loading requirements for use in the design and construction of buildings and structures with the intent to minimize hazard to life and improve the occupancy capability of essential facilities after a design level event or occurrence.

SECTION 1601
GENERAL

1601.1 Scope. The provisions of this chapter shall govern the structural design of buildings, structures and portions thereof regulated by this code.

While a significant portion of Chapter 16 is dedicated to the determination of minimum design loads, it also includes other important criteria that impact the design of structures, such as the permitted design methodologies, as well as the combinations of design loads used to establish the required minimum strength of structural members. Unless stated otherwise, the criteria found in this chapter are applicable to all buildings and structures. See Chapter 34 for application of these requirements to alterations, additions or repairs to existing structures.

SECTION 1602
DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following terms are defined in Chapter 2:

Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with structural design. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

ALLOWABLE STRESS DESIGN.
OTHER STRUCTURES.

PANEL (PART OF A STRUCTURE).

RESISTANCE FACTOR.

RISK CATEGORY.

STRENGTH, NOMINAL.

STRENGTH, REQUIRED.

STRENGTH DESIGN.

SUSCEPTIBLE BAY.

VEHICLE BARRIER.

NOTATIONS.

\( D \) = Dead load.

\( D_i \) = Weight of ice in accordance with Chapter 10 of ASCE 7.

\( E \) = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7.

\( F \) = Load due to fluids with well-defined pressures and maximum heights.

\( F_s \) = Flood load in accordance with Chapter 5 of ASCE 7.

\( H \) = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.

\( L \) = Roof live load greater than 20 psf (0.96 kN/m²) and floor live load.

\( L_r \) = Roof live load of 20 psf (0.96 kN/m²) or less.

\( R \) = Rain load.

\( S \) = Snow load.

\( T \) = Self-straining load.

\( V_{se} \) = Nominal design wind speed (3-second gust), miles per hour (mph) (km/hr) where applicable.

\( V_{up} \) = Ultimate design wind speeds (3-second gust), miles per hour (mph) (km/hr) determined from Figures 1609A, 1609B, or 1609C or ASCE 7.

\( W \) = Load due to wind pressure.

\( W_i \) = Wind-on-ice in accordance with Chapter 10 of ASCE 7.

These notations are used to refer to specific nominal loads that are determined in this chapter for use in the load combinations in Section 1605:

- \( D \) is the nominal dead load determined in Section 1606. Also see the definition of “Dead load.”
- \( D_i \) is the weight of ice. See the ASCE 7 provisions referenced in Section 1614.
- Earthquake load effect. \( E \). In Section 12.4.2 of ASCE 7 includes the effects of the horizontal load, \( E_h \) as well as a vertical component. \( E \). \( E_r \) is the product of the redundancy factor, \( \rho \), and \( Q_r \), the effects of horizontal earthquake forces. \( E_v \) accounts for vertical acceleration due to earthquake ground motion, taken as \( 0.2 S_{05} D \).

Note that its magnitude is not intended to represent a total vertical response, since that is not likely to coincide with the maximum horizontal response. It is essentially a portion of the dead load, \( D \), that is added in “additive” load combinations or subtracted in “counteractive” load combinations. The term \( S_{05} \) design spectral response acceleration at short periods, is explained in the commentary to Section 1613.3.4.

For example, when this expression is used in the alternative allowable stress design load combinations of Section 1605.3.2 that include earthquake load effects the resulting combinations are as follows:

Equation 16-21
\[
D + L + S + E/1.4 = (1 + 0.143 S_{05}) D + L + S + \rho Q_r /1.4
\]

Equation 16-22
\[
0.9D + E/1.4 = (0.9 - 0.143 S_{05}) D + \rho Q_r /1.4
\]

Earthquake design criteria is provided in Section 1613, which, in turn, references the relevant ASCE 7 provisions for computation of the earthquake load effects. While these loads are necessary for establishing the required strength, the computed forces approximate the expected deformations under the design earthquake ground motions and are not applied to a structure in an actual earthquake.

- \( F \) refers to the nominal load due to fluids having “well defined pressures and maximum heights.” Unlike most other nominal loads, there is no code section governing the determination of fluid loads. Also note that \( F \) includes a vertical component (fluid weight), as well as a horizontal component (lateral pressure).
- \( F_s \) is used to refer to the flood load that is determined under Chapter 5 of ASCE 7. Note that \( F_s \) is not explicitly included under other loads listed for the alternative ASD combination in Section 1605.3.2.
- \( H \) is used to refer to the nominal load resulting from lateral soil pressure, lateral pressure of ground water or the lateral pressure of bulk materials. Section 1610 specifies minimum requirements for lateral soil loads. Note that there are not specific provisions for the determination of load resulting from the lateral pressure of bulk materials.
- \( L \) in the nominal live load determined in accordance with Section 1607 (also see the definition of “Live load”). In addition to floor live loads, it includes roof live loads that exceed the limit on \( L_r \). \( L_r \) represents nominal roof live loads up to 20 psf (0.96 N/m²).
- \( R \) is the nominal rain load determined in accordance with Section 1611.
• S is the nominal snow load determined in accordance with Section 1608.
• T is used to refer to self-straining forces resulting from contraction or expansion due to temperature change, shrinkage, moisture change or creep, as well as movement due to differential settlement. A thermal gradient at an exterior wall is an example of a structural element where these self-straining forces can affect the design. Unlike most other nominal loads, there is no code section governing the determination of self-straining forces. T is not included directly in the load combinations, but reference to it is found in Sections 1605.2.1 and 1605.3.1.2.
• V<sub>med</sub> is the term used to refer to nominal design wind speeds that are determined in Section 1609.3.1.
• V<sub>eg</sub> is the term used to refer to the mapped wind speeds in order to differentiate them from the nominal design wind speeds.
• W is the strength-level wind load determined in accordance with Section 1609.
• W<sub>i</sub> is the wind-on-ice loading. See the ASCE 7 provisions referenced in Section 1614.

SECTION 1603 CONSTRUCTION DOCUMENTS

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 shall be indicated on the construction documents.

Exception: Construction documents for buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308 shall indicate the following structural design information:

1. Floor and roof live loads.
2. Ground snow load, P<sub>g</sub>
3. Ultimate design wind speed, V<sub>s</sub>, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V<sub>eg</sub>, as determined in accordance with Section 1609.3.1 and wind exposure.
4. Seismic design category and site class.
5. Flood design data, if located in flood hazard areas established in Section 1612.3.
6. Design load-bearing values of soils.

The term "construction documents" is defined in Chapter 2. It is commonly used to refer to calculations, drawings and specifications but it includes other data that is required to indicate compliance with the code as described in Section 107. The purpose of this section is to specifically require the design professional to provide the building official with the appropriate structural details, criteria and design load data for verifying compliance with the provisions of this chapter. Note that additional structural information and specific submittal documents may also be required to be incorporated by Chapters 17 through 23.

The construction documents are required to contain sufficient detail for the building official to perform plan review and field inspection, as well as for construction activity. Dimensions indicated on architectural drawings are not required to be duplicated on the structural drawings and vice versa. The design loads, to be indicated by the design professional on the construction documents, are to be consistent with the loads used in the structural calculations. Note that the loads are not required to be on the construction drawings but must be included within the construction documents in a manner such that the design loads are clear. The building official is to compare the loads on the construction documents with the applicable minimum required loads as specified by this chapter. The inclusion of the load design information is an indication that the structure has been designed for the loads required by the code. It should be emphasized that these requirements for construction documents are applicable regardless of the involvement of a registered design professional, which is regulated by the applicable state's licensing laws. The exception provides a less extensive list of structural data to be indicated for buildings constructed in accordance with the conventional wood-frame provisions of Section 2308. This is appropriate in view of the prescriptive nature of these requirements.

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Use of live load reduction in accordance with Section 1607.10 shall be indicated for each type of live load used in the design.

• The purpose of the requirement in this section is to provide information for the building official to facilitate the plan review process. The floor live loads, which are indicated on the construction documents by the design professional, are required to meet or exceed the loads in Section 1607. Any live load reductions taken are also to be indicated.

1603.1.2 Roof live load. The roof live load used in the design shall be indicated for roof areas (Section 1607.12).

• This section provides information allowing the building official to facilitate the plan review process. The roof live loads, indicated on the construction documents by the design professional, are required to meet or exceed the loads in Section 1607.12.

1603.1.3 Roof snow load data. The ground snow load, P<sub>g</sub>, shall be indicated. In areas where the ground snow load, P<sub>g</sub>, exceeds 10 pounds per square foot (psf) (0.479 kN/m²), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P<sub>f</sub>
2. Snow exposure factor, $C_e$
3. Snow load importance factor, $I$
4. Thermal factor, $C_r$

The roof snow load design basis, indicated on the construction documents (design drawings or specifications) by the design professional, provides information allowing the building official to facilitate the plan review process. The flat roof snow load, snow exposure factor, snow load importance factor, $I$, and roof thermal factor are not to be less than the minimum requirements established by Section 1608.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. Ultimate design wind speed, $V_{wp}$ (3-second gust), miles per hour (km/hr) and nominal design wind speed, $V_{wor}$, as determined in accordance with Section 1609.3.1.
2. Risk category.
3. Wind exposure. Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m²) to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

The wind load design basis, indicated on the construction documents (design drawings or specifications) by the design professional, provides information allowing the building official to facilitate the plan review process. All five of the indicated items are to be on the submitted construction documents. Each of the indicated items is an important parameter in the determination of the wind resistance that is required in the building framework. The building official should verify that the information is on the construction documents during the plan review process. The correctness of the listed items is the responsibility of the owner or the owner's design professional.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral force-resisting system of the structure:

1. Risk category.
2. Seismic importance factor, $I_e$
4. Site class.
5. Design spectral response acceleration parameters, $S_{DS}$ and $S_{DR}$.
6. Seismic design category.
7. Basic seismic force-resisting system(s).
8. Design base shear(s).
9. Seismic response coefficient(s), $C_p$
10. Response modification coefficient(s), $R$.
11. Analysis procedure used.

The earthquake load design basis, indicated on the construction documents by the design professional, provides information that allows the building official to facilitate the plan review process. All buildings, except those indicated in the exceptions to Section 1613.1, are to be designed for earthquake effects. The earthquake design data for a specific building are required to meet or exceed the minimum requirements established by Section 1613.

1603.1.6 Geotechnical information. The design load-bearing values of soils shall be shown on the construction documents.

Load-bearing values for soils must be documented so that the foundation design can be verified.

1603.1.7 Flood design data. For buildings located in whole or in part in flood hazard areas as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. In flood hazard areas not subject to high-velocity wave action, the elevation of the proposed lowest floor, including the basement.
2. In flood hazard areas not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
3. In flood hazard areas subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

The flood hazard elevation information to be shown on the construction documents by the registered design professional provides information that allows the building official to facilitate the plan review process. By providing the design documentation required in Section 1612.5 and by citing the specified flood information, the registered design professional is indicating that the building was designed in accordance with the flood hazard requirements in Section 1612. If any portion of a building is in a flood hazard area, then the building must meet the corresponding flood requirements.

Depending on the nature of the designated flood hazard area, certain elevation requirements are to be met. In flood hazard areas not subject to high-velocity wave action (commonly called A/BE zones), the lowest floor of all buildings and structures, or the elevation to which nonresidential buildings are dry floodproofed, is to be located at or above the elevation specified in Section 1612.4 (which references ASCE 24). In flood hazard areas subject to high-
velocity wave action (commonly called V or VE zones), the bottom of the lowest horizontal structural member is to be located at or above the elevation specified in Section 1612.4 (which references ASCE 24). These elevations are the main factor used in determining flood insurance premium rates. Constructing a building or structure with its lowest floor (or dry floodproofing) below the required elevation will result in significantly higher flood insurance premiums for the building owner.

**1603.1.8 Special loads.** Special loads that are applicable to the design of the building, structure or portions thereof shall be indicated along with the specified section of this code that addresses the special loading condition.

- Indication of special loads on the construction documents by the design professional provides information that allows the building official to facilitate the plan review process. The design professional is expected to identify any special loads that the occupancy will impose on the structure. These could include the operating weight of specialty equipment, for instance. There are also instances outside of Chapter 16 where the code specifies loading criteria that the structural design must address. For example, Section 415.8.3 requires that liquid petroleum gas facilities be in accordance with NFPA 58. In that document, a room housing a liquefied petroleum gas distribution facility must be separated from an adjacent use with ceilings and walls that are designed for a static pressure of 100 pounds per square foot (psf) (4788 Pa).

**1603.1.9 Systems and components requiring special inspections for seismic resistance.** Construction documents or specifications shall be prepared for those systems and components requiring special inspection for seismic resistance as specified in Section 1705.11 by the registered design professional responsible for their design and shall be submitted for approval in accordance with Section 107.1. Reference to seismic standards in lieu of detailed drawings is acceptable.

- This section provides for construction documents for the systems and components specified in Section 1705.11. Generally, the systems and components are the seismic-force-resisting systems, certain mechanical and electrical equipment and systems and architectural components for buildings where the seismic performance category is high. Construction documents are needed for these important items to verify that they comply with code requirements.

**SECTION 1604 GENERAL DESIGN REQUIREMENTS**

**1604.1 General.** Building, structures and parts thereof shall be designed and constructed in accordance with strength design, load and resistance factor design, allowable stress design, empirical design or conventional construction methods, as permitted by the applicable material chapters.

- This section identifies the various design methods that are permitted by the code and referenced design standards. The details for these design methods are either explicitly specified in the code or are located in the design standards that are referenced in the structural material design chapters for concrete, aluminum, masonry, steel and wood. For example, empirical design of masonry is addressed in Section 2109. The design of masonry using allowable stress design is required to be in accordance with TMS 402/ACI 530/ASCE 5 with the modifications in Section 2107.

**1604.2 Strength.** Buildings and other structures, and parts thereof, shall be designed and constructed to support safely the factored loads in load combinations defined in this code without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this code without exceeding the appropriate specified allowable stresses for the materials of construction.

- Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the building official.

- This section describes the strength and allowable stress design methods in the code. It also gives the building official approval authority for structural loads in buildings used for occupancies that are not specifically addressed in this chapter.

**1604.3 Serviceability.** Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift. See Section 12.12.1 of ASCE 7 for drift limits applicable to earthquake loading.

- The stated objective in this section is to limit member deflections and system drifts by providing adequate stiffness. Note that the definition of "Limit state" describes a serviceability limit state as a condition beyond which a structure or member is no longer useful for its intended purpose. This would be in contrast to a strength limit state beyond which the structure or member is considered unsafe. The code does not directly address a serviceability limit state. Instead the code limits certain member deflections for specific nominal loads in addition to the story drift limits for earthquake load effects.

The deflection limits for structural members must be in accordance with the requirements in this and subsequent sections. This section also provides a reference to the story drift limitations in the earthquake provisions. Generally, deflection limits are needed for the comfort of the building occupants and so that the structural member's deflection does not cause damage to supported construction. Excessive
deflection can also contribute to excessive vibration, which is discomforting to the occupants of the building.

TABLE 1604.3. See below.

The deflection limits in this table apply when they are more restrictive than those in the structural design standards that are indicated in Sections 1604.3.2 through 1604.3.5. Note that the deflection limits for exterior walls and interior partitions vary depending on the type of finish (i.e., flexible, plaster and stucco or other brittle finishes). A flexible finish is intended to be one that has been designed to accommodate the higher deflection indicated and remain serviceable. A brittle finish is any finish that has not been designed to accommodate the deflection allowed for a flexible finish. The more restrictive limit for plaster or stucco is based on ASTM C 926. The deflection limit for a roof member supporting a plaster ceiling is intended to apply only for a plaster ceiling. The limit for a roof member supporting a gypsum board ceiling is that listed in the table for "supporting a nonplaster ceiling."

The answers to the following frequently asked questions provide further guidance on applying the deflection limits of the code.

Frequently Asked Questions—Table 1604.3

Q1. For purposes of checking the deflection limits of Table 1604.3, should the calculated wind load be used directly or the combined loads from Section 1605.3.1 or 1605.3.2?

A1. In computing deflections to verify compliance with Table 1604.3 limits, the loads shown in the column headings of Table 1604.3 are the only loads that must be applied to the member. The load used to check the deflection in this case is W, the nominal wind load in accordance with Section 1609. It is not necessary to use the load combinations of Section 1605.3 for verifying that the deflection limits have been met.

Q2. Note that permits the wind load to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits. Please explain the basis for this 0.42 adjustment.

A2. There are two aspects to this adjustment. The first is the conversion from a wind speed with a 50-year mean recurrence interval (MRI) to a 10-year MRI event at an allowable stress design level. The mapped wind speeds of the code have generally been based on an MRI of 50 years. Serviceability checks, such as deflection, have typically been based on a lower MRI (for example, 10 years). The ASCE 7 commentary to the 2005 edition provided factors to convert to wind speeds with MRIs other than 50 years in

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>L</th>
<th>S or W</th>
<th>D + L&lt;sup&gt;2&lt;/sup&gt;&lt;sup&gt;0.5&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>$L_{360}$</td>
<td>$S_{360}$</td>
<td>$W_{240}$</td>
</tr>
<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>$L_{240}$</td>
<td>$S_{240}$</td>
<td>$W_{180}$</td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>$L_{180}$</td>
<td>$S_{180}$</td>
<td>$W_{120}$</td>
</tr>
<tr>
<td>Floor members</td>
<td>$L_{360}$</td>
<td></td>
<td>$W_{240}$</td>
</tr>
<tr>
<td>Exterior walls and interior partitions:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td></td>
<td>$S_{360}$</td>
<td></td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td></td>
<td>$S_{240}$</td>
<td></td>
</tr>
<tr>
<td>With flexible finishes</td>
<td></td>
<td>$W_{120}$</td>
<td></td>
</tr>
<tr>
<td>Farm buildings</td>
<td></td>
<td></td>
<td>$W_{180}$</td>
</tr>
<tr>
<td>Greenhouses</td>
<td></td>
<td></td>
<td>$W_{120}$</td>
</tr>
</tbody>
</table>

For SI: 1 foot = 304.8 mm.

a. For structural framing and siding made of formed metal sheets, the total load deflection shall not exceed 0.760. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed 0.150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed 0.760. For roofs, this exception only applies when the metal sheets have no roof covering.

b. Interior partitions not exceeding 6 feet in height and flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

c. See Section 2403 for glass supports.

d. For wood structural members having a moisture content of less than 16 percent at time of installation and used under dry conditions, the deflection resulting from \( L + 0.5D \) is permitted to be substituted for the deflection resulting from \( L + D \).

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Section 1614 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.

f. The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein.

g. For steel structural members, the dead load shall be taken as zero.

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed 0.760. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed 0.175 for each glass lite or 0.760 for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed 0.120.

i. For cantilever members, \( L \) shall be taken as twice the length of the cantilever.
Table C6-7. For wind speeds between 85 mph and 100 mph, the factor for 10 years in Table C6-7 was 0.84. This conversion factor applies to the wind speed, \( V \). Since design wind pressure is a function of \( V^2 \), the conversion factor must be squared before applying it to the design wind pressure. The factor 0.84 squared is 0.7056, which is rounded off to 0.7, which was the factor given in previous editions of the code. The second aspect is the conversion of the wind load from strength level to allowable stress design level which means the 0.7 factor is multiplied by 0.6, giving rise to the factor of 0.42 in the footnote.

Q3. In Table 1604.3, Note g states "dead load shall be taken as zero for structural steel members." Would this apply to the precomposition check of composite beam deflection limits under wet weight of concrete?

A3. No. The serviceability requirements of Section 1604.3 apply to the finished construction. The loading condition described would be a construction consideration, which is not directly regulated by the serviceability criteria.

1604.3.1 Deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Sections 1604.3.2 through 1604.3.5 or that permitted by Table 1604.3.

- The deflection of structural members is limited by the code, as well as certain material standards for damage control of supported construction and human comfort. Generally, the public equates visible deflection, or even detectable vibration, with a potentially unsafe condition (which in many cases is not true). The intent of this section is that deflection is not to exceed either the limitations in the applicable material design standard or the applicable specified requirements.

1604.3.2 Reinforced concrete. The deflection of reinforced concrete structural members shall not exceed that permitted by ACI 318.

- The deflection limitations in ACI 318 are not to be exceeded for reinforced (and prestressed) concrete (see ACI 318 for detailed deflection requirements).

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ASCE 8, SJI CJ-1.0, SJI JG-1.1, SJI K-1.1 or SJI LH/DLH-1.1, as applicable.

- The design standard to be met depends on the type of steel structural member. For example, steel joists are to meet the deflection limitations in the Steel Joist Institute's (SJI) standard, which is applicable to the type of joist, and rolled steel members are to meet the deflection criteria in AISC 360.

1604.3.4 Masonry. The deflection of masonry structural members shall not exceed that permitted by TMS 402/ACI 530/ASCE 5.

- The deflection limits for masonry beams and lintels is specified in TMS 402/ACI 530/ASCE 5.

1604.3.5 Aluminum. The deflection of aluminum structural members shall not exceed that permitted by AA ADM1.

- The deflection of aluminum structural members is limited to that in AA-ADM1 for aluminum or that permitted by Table 1604.3 of the code.

1604.3.6 Limits. The deflection limits of Section 1604.3.1 shall be used unless more restrictive deflection limits are required by a referenced standard for the element or finish material.

- The limits specified in Table 1604.3 apply to the indicated members for any structural material. As indicated in Section 1604.3.1, the deflection limits in the structural material standards apply when they are more restrictive than indicated in the table.

The deflection limits that are applicable to a flat concrete roof member that does not support any non-structural elements likely to be damaged by large deflections are summarized in Table 1604.3.6. As can be seen, the code deflection criteria is more stringent and would govern the design of this type of member.

<table>
<thead>
<tr>
<th>Table 1604.3.6 CONCRETE ROOF MEMBER DEFLECTION CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>L</td>
</tr>
<tr>
<td>D + L</td>
</tr>
<tr>
<td>IBC Table 1604.3</td>
</tr>
<tr>
<td>Roof members</td>
</tr>
<tr>
<td>Concrete per ACI 318 See Table 9.5(b)</td>
</tr>
<tr>
<td>Flat roofs</td>
</tr>
<tr>
<td>a. No deflection limit specified in ACI 318.</td>
</tr>
</tbody>
</table>

1604.4 Analysis. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.
The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. Except where diaphragms are flexible, or are permitted to be analyzed as flexible, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

This section includes the general requirements for structural analysis. The principles stated in this section are those commonly found in structural engineering textbooks. The requirement that structural analysis be capable of demonstrating a complete load path is essential to the adequate resistance of the structural system to wind loads or earthquake effects. The load path is to be capable of transferring all of the loads from their point of application onto the structure to the foundation. It is also important that nonstructural rigid elements be properly accounted for in the design. For example, a partial-height rigid masonry wall placed between steel columns in a steel frame will resist the horizontal shear load and cause bending in the column unless a flexible joint is provided between the wall and the column.

The definition of "Diaphragm" in Section 202.1 includes definitions that distinguish a rigid diaphragm from a flexible diaphragm. The distribution of forces in buildings with flexible diaphragms differs from those having rigid diaphragms. Where the diaphragm is determined to be flexible, the effect of diaphragm rigidity on the distribution of lateral forces is considered to be negligible and can therefore be neglected in the structural analysis. Otherwise, for structures having rigid diaphragms, this section requires the engineer to distribute lateral forces to the vertical supporting elements in proportion to their rigidities, and to include the effect of the increased forces induced on the vertical supporting elements resulting from torsion due to eccentricity between the center of mass and the center of rigidity.

1604.5 Risk category. Each building and structure shall be assigned a risk category in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the risk category shall not be taken as lower than the occupancy category specified therein.

This section requires classification of the risk category of any building in accordance with the nature of occupancy as described in Table 1604.5. The risk category serves as a threshold for a variety of code provisions related to earthquake, flood, snow and wind loads. Particularly noteworthy are the importance factors that are used in the calculation of design earthquake, snow and wind loads. The value of the importance factor generally increases with the importance of the facility. Structures assigned greater importance factors must be designed for larger forces. The result is a more robust structure that would be less likely to sustain damage under the same conditions than a structure with a lower importance factor. The intent is to enhance a structure's performance based upon its use or the need to remain in operation during and after a design event.

The impact of a higher risk category classification is not limited to increasing the design loads. Compared to Risk Category I, II or III, for instance, a Risk Category IV classification can lead to a higher seismic design category classification that can, in turn, require more stringent seismic detailing and limitations on the seismic-force-resisting system. This can also affect the seismic design requirements for architectural, mechanical and electrical components and systems.

**TABLE 1604.5.** See page 16-9.

The risk category determined in this table generally increases with the importance of the facility, which relates to the availability of the facility after an emergency, and the consequence of a structural failure on human life. The categories range from Risk Category I, which represents the lowest hazard to life, through Risk Category IV, which encompasses essential facilities.

**Risk Category IV:** These are buildings that are considered to be essential in that their continuous use is needed, particularly in response to disasters. Fire, rescue and police stations, and emergency vehicle garages must remain operational during and after major events, such as earthquakes, floods or hurricanes. The phrase "designated as essential facilities" refers to designation by the building official that certain facilities are required for emergency response or disaster recovery. This provides jurisdictions the latitude to identify specific facilities that should be considered essential in responding to various types of emergencies. These could include structures that would not otherwise be included in this risk category. This designation would only be made with consideration of broader public policy, as well as emergency preparedness planning within the jurisdiction in question. The reasons for including facilities, such as hospitals, fire stations, police stations, emergency response operations centers, etc., should be self-evident. Some items warranting additional discussion are as follows:

- Designated emergency shelters and designated emergency response facilities. These items repeat the term "designated," which is referring to designation by the building official that the facilities have been identified as necessary for sheltering evacuees or responding to
emergencies (see discussion of "designated-above"). For example, an elementary school having an occupant load of 275 would typically be considered a Risk Category III facility. If that school is designated as an emergency shelter, then the school will be considered a Risk Category IV building.

- **Facilities supplying emergency backup power for Risk Category IV.** A power-generating station or other utility (such as a natural gas facility) is to be classified as Risk Category IV only if the facility serves an emergency backup function for a Risk Category IV building, such as a fire station or police station. Otherwise, the power-generating station or utility should be classified as Risk Category III.

- **Structures with quantities of highly toxic materials in excess of the quantities permitted for a control area in Table 307.1(2).** This applies only to "Highly toxic materials" (see definition in Section 307.2), which are covered in the second row of Table 307.1(2). That table lists the maximum allowable quantities per control area of materials posing a health hazard. Since the use of control areas is permitted by Table 307.1(2), it is reasonable to recognize the control area for the purpose of making this risk category determination. In other words, this applies to occupancies

---

**TABLE 1604.5**

RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>- Agricultural facilities.</td>
</tr>
<tr>
<td></td>
<td>- Certain temporary facilities.</td>
</tr>
<tr>
<td></td>
<td>- Minor storage facilities.</td>
</tr>
<tr>
<td>II</td>
<td>Buildings and other structures except those listed in Risk Categories I, III and IV.</td>
</tr>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>- Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>- Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>- Any other occupancy with an occupant load greater than 5,000.</td>
</tr>
<tr>
<td></td>
<td>- Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Risk Category IV.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:</td>
</tr>
<tr>
<td></td>
<td>Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the International Fire Code; and</td>
</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released.</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings and other structures designed as essential facilities, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>- Group I-2 occupancies having surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>- Fire, rescue, ambulance and police stations and emergency vehicle garages.</td>
</tr>
<tr>
<td></td>
<td>- Designated earthquake, hurricane or other emergency shelters.</td>
</tr>
<tr>
<td></td>
<td>- Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.</td>
</tr>
<tr>
<td></td>
<td>- Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures containing quantities of highly toxic materials that:</td>
</tr>
<tr>
<td></td>
<td>Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the International Fire Code; and</td>
</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released.</td>
</tr>
<tr>
<td></td>
<td>- Aviation control towers, air traffic control centers and emergency aircraft hangars.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures having critical national defense functions.</td>
</tr>
<tr>
<td></td>
<td>- Water storage facilities and pump structures required to maintain water pressure for fire suppression.</td>
</tr>
</tbody>
</table>

---

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.
that are classified as Group H-4 based on the quantities of highly toxic material exceeding the permitted quantity within a control area. However, recognizing control areas means that both the risk category as well as the occupancy classification could be lowered by adding either fire-resistance-rated walls or floor/ceiling assemblies in order to divide the building into a number of smaller control areas. The additional wording "...sufficient to pose a threat to the public if released" places a further qualification on the material quantity, but it is subjective since the threat to the public could be difficult to determine. Also note that a Group H-4 occupancy classification could be based on exceeding the quantities permitted for toxics or corrosives (see Table 307.1(2)), but the presence of those materials would not affect the assessment of the facility as Risk Category IV.

Risk Category III: Risk Category III buildings include those occupancies that have relatively large numbers of occupants because of the overall size of the building. They also include uses that pose an elevated life-safety hazard to the occupants, such as public assembly, schools or colleges. In addition, Risk Category III includes uses where the occupant's ability to respond to an emergency is either restricted, such as in jails, or otherwise impaired, such as in nursing homes housing patients that require skilled nursing care. A discussion of some of the specific table listings follows:

- **Buildings and other structures with a primary occupancy that is public assembly with an occupant load greater than 300.** Public assembly occupancies meeting this criterion will typically be classified as Group A in Chapter 3. The wording requires agreement on the determination that a building's "primary occupancy" is in fact public assembly. This could be as simple as verifying that the portion of the building housing the public assembly occupancy is more than 50 percent of the total building area.

- **Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities.** This category applies to health care facilities with at least 50 resident patients. The term "resident patient" is not defined or used elsewhere in the code, but would seem to refer to locations where those patients receive around-the-clock (24-hour) care as opposed to ambulatory surgery centers or outpatient units. This table entry covers facilities where patients have difficulty responding to an emergency or are incapable of self-preservation.

- **Buildings having an occupant load greater than 5,000.** Uses that pose elevated life-safety concerns, such as public assembly uses, schools and health care facilities, are covered elsewhere and have a much lower threshold based on the number of occupants. This table entry covers buildings that are large enough to have more than 5,000 occupants, providing added protection for the occupants of larger structures whatever the use happens to be. In order to determine occupant load, the methods outlined in Section 1004 are normally used. Chapter 10 sets forth standards that provide a reasonably conservative number of occupants for all spaces, and while actual loads are commonly less than the design amount, it is not unusual in the life of a space in a building to have periods when high actual occupant loads exist. Because there is no clear rationale that connects the occupant load used to calculate minimum means of egress requirements to the risks associated with structural design, Note A provides some reasonable adjustment to this determination by permitting the use of net floor area. It provides a more reasonable approach for occupancies, such as office, mercantile and residential, that are required to base occupant load on gross area—an area that includes corridors, stairways, elevators, closets, accessory areas, structural walls and columns, etc.

- **Power-generating stations, potable water treatment facilities, wastewater treatment and other public utilities not included in Risk Category IV.** A failure and subsequent shutdown of these types of facilities would not pose an immediate threat to life safety. These infrastructure items are considered Risk Category III because of the impact an extended disruption in service can have on the public.

- **Buildings not included in Risk Category IV containing quantities of toxic or explosive materials that exceed permitted quantities per control area in Table 307.1(1) or 307.1(2) and are sufficient to pose a threat to the public if released.** Buildings included under Risk Category IV would be those containing quantities of highly toxic materials that exceed the permitted quantity in Table 307.1(2) (see discussion under "Risk Category IV" above). This item addresses buildings with explosives or toxic substances, both of which are defined in Section 307.2.

Risk Category II: Risk Category II buildings represent a lesser hazard to life because of fewer building occupants and smaller building size compared to those that are considered Risk Category III. Since Risk Categories III and IV represent buildings with higher risk or essential facilities, on a relative scale Risk Category II can be thought of as a "standard occupancy" building as evidenced by importance factors for earthquake, snow and wind that are all equal to 1.

Risk category I: Risk Category I buildings exhibit the lowest hazard to life since they have little or no
human occupants or, for those that are temporary, the exposure to the hazards of earthquakes, floods, snow and wind would be considerably less than that of a permanent structure. Note that this category includes "minor storage facilities," but the code does not provide an explanation of which storage facilities could be considered minor.

1604.5.1 Multiple occupancies. Where a building or structure is occupied by two or more occupancies not included in the same risk category, it shall be assigned the classification of the highest risk category corresponding to the various occupancies. Where buildings or structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a building or structure provides required access to, required egress from or shares life safety components with another portion having a higher risk category, both portions shall be assigned to the higher risk category.

- Buildings are frequently occupied by a mixture of uses or occupancies. A single-use building is probably the exception rather than the rule. Where multiple occupancies are proposed in a building, the risk category of each one must be considered. In some cases, the proposed occupancies in a building will fall into more than one risk category and the requirements for multiple occupancies stated in Section 1604.5.1 must be satisfied. These requirements were previously part of the earthquake load provisions. As of the 2006 edition, they were relocated so that they now apply regardless of the type of load being considered. The code identifies two design options for mixed-use buildings. The entire structure can be designed as a single unit based on the requirements for the most stringent risk category for the building. Alternatively, the engineer can structurally separate portions of the structure containing distinct occupancy categories and design each portion accordingly based on its risk category.

This section also provides direction regarding access to and egress from adjacent structures that fall into different occupancy categories by making the requirements for the more stringent risk category applicable to both structures. This requirement is the result of lessons learned from events, such as earthquakes, in which essential functions have been rendered unusable because of a failure in an adjacent structure.

1604.6 In-situ load tests. The building official is authorized to require an engineering analysis or a load test, or both, of any construction whenever there is reason to question the safety of the construction for the intended occupancy. Engineering analysis and load tests shall be conducted in accordance with Section 1709.

- The building official has the option of requiring either a structural analysis, an on-site in-situ load test, or both, in accordance with Section 1714, on an existing structure, building or portion thereof if there is reasonable doubt as to structural integrity. The building official should document his or her reasons for the testing requirement. Whenever possible, the concern should be addressed by structural analysis since load testing a structure is very expensive. One example would be an analysis by a third-party engineering firm acceptable to both the building official and the owner. The structural integrity may be examined for items such as visible signs of excessive settlement or lateral deflection, such as cracks in concrete foundation walls or excessive vibration when the assembly is loaded. The procedure must simulate the actual load conditions to which the structure is subjected during normal use.

1604.7 Preconstruction load tests. Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable referenced standards, or alternative test procedures in accordance with Section 1707, shall be load tested in accordance with Section 1710.

- The alternative test procedure described in Section 1707 and the preconstruction load test procedure described in Section 1710 are intended to apply to materials or an assembly of structural materials that do not have an accepted analysis technique; thus, they are approved for use by way of the alternative test procedure. The preconstruction test procedure in Section 1710 includes the determination of the allowable superimposed design load (see Section 1710 for details).

1604.8 Anchorage. Buildings and other structures, and portions thereof, shall be provided with anchorage in accordance with Sections 1604.8.1 through 1604.8.3, as applicable.

- Sections 1604.8.1 through 1604.8.3 contain general anchorage requirements as well as specific requirements for the anchorage of walls and decks.

1604.8.1 General. Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed loads.

- This section states a specific load path requirement for anchorage that is already required in more general terms by Section 1604.4. Required anchorage resistance to uplift and sliding is determined in accordance with the appropriate load combinations in Section 1605, which account for the effects of earthquakes, fluid pressures, lateral earth pressure or wind (also see Section 1604.9).

An example of the condition described in this section is roof uplift as a result of high winds. Table 2306.10.1 requires roof uplift connectors for wood-frame construction designed and installed in accordance with the prescriptive requirements for conventional light-frame construction. Note that net roof uplift occurs at all wind speeds indicated in the table; thus, uplift connectors are required for roof wood framing for all of the indicated wind speed locations.

1604.8.2 Structural walls. Walls that provide vertical loadbearing resistance or lateral shear resistance for a portion of the structure shall be anchored to the roof and to all floors and
members that provide lateral support for the wall or that are supported by the wall. The connections shall be capable of resisting the horizontal forces specified in Section 1.4.4 of ASCE 7 for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of structures assigned to all other seismic design categories.  

Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and 1613 for earthquake design requirements.

- This section refers to the ASCE 7 technical provisions for the minimum strength design load on anchorages of walls to the horizontal diaphragms that provide lateral support. The connection requirements apply to both bearing walls and walls that resist lateral loads. The wall anchorage must be a positive connection that does not rely on friction for load transfer. Earthquake detailing provisions may result in higher anchorage design forces based on the seismic design category of the building, as well as the mass of the wall.

1604.8.3 Decks. Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. Connections of decks with cantilevered framing members to exterior walls or other framing members shall be designed for both of the following:

1. The reactions resulting from the dead load and live load specified in Table 1607.1, or the snow load specified in Section 1608, in accordance with Section 1605, acting on all portions of the deck.

2. The reactions resulting from the dead load and live load specified in Table 1607.1, or the snow load specified in Section 1608, in accordance with Section 1605, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the deck.

- This requirement for the positive anchorage of decks is a result of failures that have occurred primarily on nonengineered residential decks. Toenail connections are very weak since many of the field connections split the wood framing member. Nails that are installed in line with applied tension forces pull out easily. Wood deck framing could be attached to a supporting ledger board by joist hangers that provide nails loaded in shear for the vertical and horizontal loads.

In order to make the loading considerations perfectly clear for decks with cantilevers, this section states the two loading conditions that are applicable. The second condition covers the case where uplift can occur at the point of connection between the wood framing and the exterior wall for cantilevered deck framing. This highlights the need for resistance to uplift by a positive connection at the exterior wall.

These conditions also clarify that snow load should be considered.

1604.9 Counteracting structural actions. Structural members, systems, components and cladding shall be designed to resist forces due to earthquakes and wind, with consideration of overturning, sliding and uplift. Continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force.

- This section requires that all elements of the structure be designed to resist the required earthquake and wind effects. The required level of resistance will be based on the appropriate load combinations of Section 1605. The term "counteracting" used in the section title refers to so-called counteracting load combinations as opposed to those in which the load effects are additive. The effects of overturning, uplift and sliding are most pronounced in these counteractive load combinations. These would consist of the following depending on the design method chosen:

- Load and resistance factor design or strength design, Section 1605.2:
  \[ 0.9D + 1.0 W + 1.6H \]  
  \[ 0.9(D + F) + 1.0E + 1.6H \]

  Equation 16-6
  Equation 16-7

- Basic allowable stress design load combinations, Section 1605.3.1:
  \[ 0.6D + 0.6W + H \]
  \[ 0.6(D + F) + 0.6E + H \]

  Equation 16-12
  Equation 16-13

- Alternative allowable stress design load combinations, Section 1605.3.2:
  \[ 0.9D + E/1.4 \]

  Equation 16-22

  (Note the adjustment to the vertical component of earthquake load effect that is permitted for evaluating foundations.)

  \[ (\gamma)D + (0.6 \times 1.3)W \]

  Equation 16-18

  (Equation 16-18 with L = 0 in accordance with Section 1605.1 requirement to investigate each load combination with one or more variable loads equal to zero, two-thirds of the dead load in accordance with requirement in Section 1605.3.2 and w set equal to 1.3 for 3-second gust wind speed.)

As can be seen, for the critical counteracting load combinations, resistance to overturning, uplift and sliding is largely provided by dead load. Note that Section 1605.3.2 requires the designer to consider only the dead load likely to be in place during a design wind event when using the alternative allowable stress load combinations. Regardless of the design methodology utilized, the designer should be cautious in estimating dead loads that resist overturning, sliding and uplift. In order to remain conservative, estimated dead load should be no more than the actual dead load in these cases (see design dead load in Section 1606.2). To keep a structure from sliding horizontally, the dead load must generate enough friction at the base of the structure to resist the hori-
zontal base shear due to earthquake and wind. Otherwise, adequate anchor-age must be provided to resist the base shear.

1604.10 Wind and seismic detailing. Lateral force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7, excluding Chapter 14 and Appendix 11A, even when wind load effects are greater than seismic load effects.

- This section is needed to clarify that the seismic detailing requirements of the code always apply, even where the wind load effects are higher than the seismic load effects. This is required because the calculated earthquake load is dependent upon an assumed level of ductility to be provided in the selected seismic-force-resisting system. If the designer does not follow through by providing the detailing required to achieve the ductility of the selected system, the assumption is invalid.

For example, consider the case where the earthquake load provisions apply in accordance with Section 1613.1, the seismic-force-resisting system is an ordinary concentrically braced steel frame and, based on the seismic design category, Section 2205 requires that the AISC 341 seismic provisions are to be met. For this case, the requirements in Section 14 of AISC 341 must be satisfied in the design of the steel frame, even if the wind load effects exceed the seismic load effects.

SECTION 1605
LOAD COMBINATIONS

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist:

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2;

2. The load combinations specified in Chapters 18 through 23; and

3. The seismic load effects including overstrength factor in accordance with Section 12.4.3 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the seismic load effects including overstrength factor in accordance with Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

1. The basic combinations for strength design with overstrength factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.

2. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-12, 16-14 and 16-16 in Section 1605.3.1.

3. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-21 and 16-22 in Section 1605.3.2.

- Generally, there are two types of load combinations specified in the code: those to be used with a strength design or load and resistance factor design (LRFD) and those to be used with allowable stress design. Where additional load combinations are specified in the structural material chapters, they apply also. Note that this section also requires that for a given load combination engineers also consider additional load cases where one or more variable loads are not acting concurrently with other variable loads. It is necessary to explore such possibilities, since they can at times result in the most critical load effect for some elements of a structure. According to the definition of “Loads,” loads that are not considered permanent are variable loads.

The load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 only apply where specifically required by the seismic provisions. They do not replace the applicable load combinations for strength design or allowable stress design. They constitute an additional requirement that must be considered in the design of specific structural elements.

The purpose of these load combinations with overstrength factor is to account for the maximum earthquake load effect, $E_{mr}$ which considers “system overstrength.” This system characteristic is accounted for by multiplying the effects of the lateral earthquake load by the overstrength factor, $\Omega_p$, for the seismic-force-resisting system that is utilized. It is representative of the upper bound system strength for purposes of designing nonyielding elements for the maximum expected load. Under the design earthquake ground motions, the forces generated in the seismic-force-resisting system can be much greater than the prescribed seismic design forces. If not accounted for, the system overstrength effect can cause failures of structural elements that are subjected to these forces. Because system overstrength is unavoidable, design for the maximum earthquake force that can be developed is warranted for certain elements. The intent is to provide key elements with sufficient overstrength so that inelastic (ductile) response/behavior appropriately occurs within the vertical-resisting elements.

The ASCE 7 earthquake load provisions that require consideration of the load combinations with over-strength factor are stated in this section. Note that these load combinations are not general load cases, but are only to be applied where specified in the indicated earthquake load provisions (see above) or in the structural materials requirements (see Sec-
tion 1810.3.11.2 for an example of the latter).

For example, Section 12.3.3.3 of ASCE 7 applies to structural elements that support discontinuous frames or shear wall systems where the discontinuity is severe enough to be deemed a structural irregularity. See Figure 1605.1(1) for examples of building configurations that include discontinuous vertical systems. Also see the commentary to Section 12.3.3.3 of ASCE 7.

Section 12.10.2.1 of ASCE 7 applies to collector elements in structures assigned to Seismic Design Category C or higher. The term "collector" describes an element used to transfer forces from a diaphragm to the supporting element(s) of the lateral-force-resisting system where the length of the vertical element is less than the diaphragm dimension at that location. Figure 1605.1(2) illustrates this relationship. Failures of collectors can result in loads not being delivered to resisting elements and separation of a portion of a building from its lateral-force-resisting system. Where a collector also supports gravity loads, such a failure would result in the loss of a portion of the gravity load-carrying system. The intent is to provide collector elements that have sufficient overstrength so that any inelastic behavior appropriately occurs within the seismic-force-resisting system rather than the collector elements.

Maximum earthquake load effect, \( E_{\text{req}} \), in Section 12.4.3 of ASCE 7 includes effects of the horizontal load, \( E_{\text{h}} \), as well as a vertical component, \( E_{\text{v}} \). \( E_{\text{h}} \) is the product of the overstrength factor, \( \Omega_{\text{v}} \), and \( Q_{\text{h}} \), the effects of horizontal earthquake forces. \( E_{\text{v}} \) is the same vertical component used for the earthquake load effect, \( E \) (see commentary, Section 1602.1).

![Figure 1605.1(2) COLLECTORS](image1)

![Figure 1605.1(1) EXAMPLES OF DISCONTINUOUS SHEAR WALLS](image2)
1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is being verified, use of the load combinations specified in Section 1605.2 or 1605.3 shall be permitted. Where the load combinations specified in Section 1605.2 are used, strength reduction factors applicable to soil resistance shall be provided by a registered design professional. The stability of retaining walls shall be verified in accordance with Section 1807.2.3.

This section permits soil resistance and strength reduction factors to be considered where strength design factored loads are used in foundation design.

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

\[ 1.4(D + F) \]  
\[ 1.2(D + F) + 1.6(L + H) + 0.5(L, \text{ or } S \text{ or } R) \]  
\[ 1.2(D + F) + 1.6(L, \text{ or } S \text{ or } R) + 1.6H + (f_r L \text{ or } 0.5W) \]  
\[ 1.2(D + F) + 1.0W + f_r L + 1.6H + 0.5(L, \text{ or } S \text{ or } R) \]  
\[ 1.2(D + F) + 1.0E + f_r L + 1.6H + f_r S \]  
\[ 0.9D + 1.0W + 1.6H \]  
\[ 0.9(D + F) + 1.0E + 1.6H \]

where:

\( f_r = 1 \) for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m²), and parking garages; and 0.5 for other live loads.

\( f_r = 0.7 \) for roof configurations (such as saw tooth) that do not shed snow off the structure, and 0.2 for other roof configurations.

Exceptions:

1. Where other factored load combinations are specifically required by other provisions of this code, such combinations shall take precedence.

2. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.9 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.

This section lists the load combinations for strength design or load and resistance factor design methods. See the definitions in Section 1602 for an explanation of the load notations. The basis for these load combinations is Section 2.3.2 of ASCE 7. These combinations of factored loads are the agreed-upon strength limit states that establish the required strength to be provided in the structural component being designed. In spite of the precise appearance of the load and resistance factor design load combinations, one should keep in mind their probabilistic nature. The goal is to allow a wide variety of structures to be designed economically with an acceptably low probability that the strength of the structure will be exceeded. Doing so necessitates combining loads in scenarios that are likely to occur. Dead load is a permanent load and it appears in every combination. The load combinations are constructed by adding the dead load to one of the variable loads at its maximum value, which is typically indicated by the load factor of 1.6. In addition, other variable loads are included with load factors that are less than 1.0. Those so-called "companion loads" represent arbitrary point-in-time values for those loads. The exception is the maximum earthquake load that has a load factor of 1.0 (see the commentary to the definitions "Factored load" and "Nominal load" in Section 202).

1605.2.1 Other loads. Where flood loads, \( F_w \), are to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Where self-straining loads, \( T \), are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.3.5 of ASCE 7. Where an ice-sensitive structure is subjected to floods due to atmospheric icing, the load combinations of Section 2.3.4 of ASCE 7 shall be considered.

Under a cooperative agreement with FEMA, American Society of Civil Engineers completed an extensive analysis of flood loads and flood load combinations. The results of this study were first included in the 1998 edition of ASCE 7. For all buildings and structures located in flood hazard areas, this section specifies that flood load and flood load combinations using the strength design method are to be determined in accordance with Section 2.3.3 of ASCE 7, which states:

2.3.3. Load Combinations Including Flood Load. When a structure is located in a flood zone (Section 5.3.1), the following load combinations shall be considered in addition to the basic combinations in Section 2.3.2:

1. In V Zones or Coastal A Zones, \( 1.0W \) in combinations (4) and (6) shall be replaced by \( 1.0W + 2.0F_w \).

2. In noncoastal A Zones, \( 1.0W \) in combination (4) and (6) shall be replaced with \( 0.5W + 1.0F_w \).

1605.3 Load combinations using allowable stress design.  
1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

\[ D + F \]  
\[ D + H + F + L \]  
\[ D + H + F + (L, \text{ or } S \text{ or } R) \]
STRUCTURAL DESIGN

\[ D + H + F + 0.75(L) + 0.75(L, \text{or} \ S \text{ or} \ R) \]  
\[ \text{Equation 16-11} \]

\[ D + H + F + (0.6W + 0.7E) \]  
\[ \text{Equation 16-12} \]

\[ D + H + F + 0.75(0.6W) + 0.75L + 0.75(L, \text{or} \ S \text{ or} \ R) \]  
\[ \text{Equation 16-13} \]

\[ D + H + F + 0.75 (0.7 E) + 0.75 L + 0.75 S \]  
\[ \text{Equation 16-14} \]

\[ 0.6D + 0.6W + H \]  
\[ \text{Equation 16-15} \]

\[ 0.6(D + F) + 0.7E + H \]  
\[ \text{Equation 16-16} \]

**Exceptions:**

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.

2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.

3. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.8 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.

4. In Equation 16-15, the wind load, \( W \), is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.

5. In Equation 16-16, 0.6 \( D \) is permitted to be increased to 0.9 \( D \) for the design of special reinforced masonry shear walls complying with Chapter 21.

See Section 1602 for an explanation of the notations. These basic load combinations for allowable stress design are based on Section 2.4.1 of ASCE 7. Note that a 0.75 factor is applied where these combinations include more than one variable load. This reduces the combined effect of these variable loads in recognition of the lower probability that two or more variable loads will reach their maximum values simultaneously.

Previous editions of the model codes specified that the overturning moment and sliding due to wind load could not exceed two-thirds of the dead load stabilizing moment; however, it was not typically applied to all elements in the building. In the code this limitation on dead load is accomplished through the load combinations. The applicable combination is \( 0.6D + 0.6W + H \). This load combination limits the dead load resisting wind loads to 60 percent \( (L_d = 0.67; \text{round down}) \) but it applies to all elements. In this form, it is required that this safety factor on dead load applies to all actions where dead load is resisting wind loads.

The load combination, \( 0.6(D + F) + 0.7E + H \), applies throughout the structure and provides for overall stability of the structure similar to the load combination for wind. In determining seismic load effect, \( E \), the definition of effective seismic weight requires inclusion of a portion of roof snow under certain conditions. Frequently the question of whether it is logical to include a component of snow load in determining \( E \), but not to use that same weight of snow in the load case to resist the overturning or uplift due to \( E \) is asked. This apparent inconsistency has been built into the seismic requirements for some time. It introduces a bit more conservatism into the earthquake loading where heavier snow loads are concerned.

Exception 1 for crane hook loads has been a long-standing allowance in the alternative basic load combinations of Section 1605.3.1 under the legacy model codes and is now permitted in the code. It allows special consideration in combining crane loads with wind, snow and roof live loads due to a lower probability that these maximum loads occur simultaneously. As noted above, these basic load combinations are based on ASCE 7 allowable stress load combinations, but there is no corresponding exception for crane loads under ASCE 7. In the development of the 2000 edition of this code, this exception was added to the basic load combinations in an attempt to provide "parity" with the alternative basic load combinations. It is worth noting that these two sets of load combinations come from two different sources as described above and, from an overall perspective, they never have provided parity. Crane loads are considered live loads (see Section 1607.13). Exception 1 has no effect on dead load, other floor live loads, rain loads or earthquake loads; thus, where crane live loads are to be considered, the exception eliminates the roof live load, \( L_d \) altogether and reduces the snow load to 0.75\( S \) and wind load to 0.5(0.6\( W \)). This exception does not negate the need to combine live loads other than the crane live loads in the usual manner with wind and snow loads. In other words, the load combinations in Equations 16-11 and 16-13 must be investigated without the crane live load and then modified versions of these combinations that include crane live load (use \( L_d \) to denote crane live load) would be considered to make use of the exception as follows:

\[ \text{Equation 16-11} \]

\[ D + H + F + 0.75(L) + 0.75(L, \text{or} \ S \text{ or} \ R) \]

\[ \text{Equation 16-11c} \]

\[ D + H + F + 0.75 (L + L_d) + 0.75 (0.75S \text{ or} \ R) \]

\[ \text{Equation 16-13} \]

\[ D + H + F + 0.75 (0.6W) + 0.75L + 0.75 \]

\[ (L, \text{or} \ S \text{ or} \ R) \]

\[ \text{Equation 16-14} \]

\[ D + H + F + 0.75 (0.5 \times 0.6W) + 0.75(L + L_d) + 0.75 (0.75S \text{ or} \ R) \]

\[ \text{Equation 16-13} \]

\[ D + H + F + 0.75 (L + L_d) + 0.75 (0.75S) \]

\[ \text{Equation 16-14} \]

\[ D + H + F + 0.75 (0.7E) + 0.75(L + L_d) + 0.75 (0.75S) \]
Exception 2 states that "flat roof snow loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads..." This is an exception to the requirement to combine the effects of snow and earthquake load as would otherwise be required by Equation 16-14.

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that increases shall be permitted in accordance with Chapter 23.

- Some code sections and some referenced standards permit increases in the allowable stress where load combinations include either wind or earthquake loads. Such increases are expressly prohibited with the basic allowable stress load combinations. This prohibition is due to the 0.75 reduction applied to multiple variable loads in these load combinations, which has replaced the concept of allowable stress increases. Several adjustment factors for the design of wood construction allow increases to reference design values, such as the load duration factor, size factor, flat use factor, repetitive member factor, buckling stiffness factor and bearing area factor. These wood adjustment factors are considered to be material dependent. Since they are not related to the presence of particular design loads, this section recognizes their use.

1605.3.1.2 Other loads. Where flood loads, \( F_e \), are to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used. Where self-straining loads, \( T \), are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.4.3 of ASCE 7 shall be considered.

- The study discussed in the commentary to Section 1605.2.1 determined flood load and flood load combination factors for use with the allowable stress method. This section specifies that flood load and flood load combinations using the allowable stress design method are to be determined in accordance with Section 2.4.2 of ASCE 7, which states:

2.4.2. Load Combinations Including Flood Load. When a structure is located in a flood zone, the following load combinations shall be considered in addition to the basic combinations in Section 2.4.1:

1. In V zones or Coastal A zones (see Section 5.3.1), \( 1.5F_e \) shall be added to other load combinations (5), (6) and (7), and \( E \) shall be set equal to zero in (5) and (6).

2. In noncoastal A zones, \( 0.75F_e \) shall be added to the combinations (5), (6) and (7), and \( E \) shall be set at zero in (5) and (6).

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. When using allowable stresses which have been increased or load combinations which have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 of ASCE 7, the coefficient \( (\alpha) \) in the following equations shall be taken as 1.3. For other wind loads, \( (\alpha) \) shall be taken as 1. When allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, \( (\alpha) \) shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, \( E_v \), in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

\[
\begin{align*}
D + L + (L, or S or R) & \quad \text{(Equation 16-17)} \\
D + L + 0.6 \alpha W & \quad \text{(Equation 16-18)} \\
D + L + 0.6 \alpha W + S2 & \quad \text{(Equation 16-19)} \\
D + L + S + 0.6 \alpha W2 & \quad \text{(Equation 16-20)} \\
D + L + S + E1.4 & \quad \text{(Equation 16-21)} \\
0.9D + E1.4 & \quad \text{(Equation 16-22)}
\end{align*}
\]

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.

2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.

- These are alternative load combinations to those in Section 1605.3.1 for use with allowable stress design. They are based on the allowable stress load combinations in the Uniform Building Code® (UBC®).

Note that the dead load is limited to two-thirds of the dead load likely to be in place during a design wind event. Section 1604.9 requires that all elements of a building be anchored to resist overturning, uplift and sliding. Often, a considerable portion of this resistance is provided by the dead load of a building, including the weight of foundations and any soil directly above them. This section requires the designer to give consideration to the dead load used to resist wind loads by stating that only the minimum
dead load likely to be in place during a design wind event is permitted to be used. This, however, does not imply that certain parts or elements of a building are not designed to remain in place. This criteria simply cautions the designer against using dead loads that may not be installed, or in place, such as can occur where a design includes an allowance for a planned future expansion.

Exception 1 for crane hook loads has been a longstanding allowance under the legacy model code styles that were also the source of these alternative basic load combinations. It allows special consideration in combining crane live loads with wind, snow and roof live loads due to a lower probability that these maximum loads occur simultaneously.

Exception 2 states that "flat roof snow loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads...". This is an exception to the requirement to combine the effects of snow and earthquake load as would otherwise be required by Equation 16-21.

Also see the commentary to Section 1602.1 for discussion of earthquake load effect, $E$, as well as other loads required by these load combinations.

**1605.3.2.1 Other loads.** Where $F$, $H$ or $T$ are to be considered in the design, each applicable load shall be added to the combinations specified in Section 1605.3.2. Where self-straining loads, $T$, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

See Section 1602 for an explanation of the notation used in this section. As indicated, the applicable loads are to be added to each of the load combinations in Section 1605.3.2.

**SECTION 1606 DEAD LOADS**

**1606.1 General.** Dead loads are those loads defined in Section 1602.1. Dead loads shall be considered permanent loads.

The nominal dead load, $D$, is determined in accordance with Section 1606. Similar to the general section on live loads (see Section 1607.1), this section provides a reminder that the term "dead loads" is defined, establishing exactly what should be considered a dead load. It also states that dead loads are considered permanent. This affects how dead loads are classified (permanent versus variable), which is a necessary distinction to make when applying the provisions for load combinations.

**1606.2 Design dead load.** For purposes of design, the actual weights of materials of construction and fixed service equipment shall be used. In the absence of definite information, values used shall be subject to the approval of the building official.

When determining the design dead loads, the actual weights of all materials, equipment, construction, etc., are to be used for the structural design. Often the exact type of materials and equipment to be installed has not yet been determined by the design team at the time the structural design is initiated. Where the actual weights are not known, it is common to use an estimate of the anticipated materials and equipment. While the actual dead load itself does not vary, the estimate of the dead load that is used in structural computations can vary. Such estimates of dead loads are typically greater than the actual dead loads so that the computations are conservative and a redesign of the structure will not be necessary when the actual weights become known. This section clarifies that the building official must approve these estimated dead loads.

Overestimating the actual dead load, so that the design computations are conservative, is acceptable when considering load combinations in Section 1605 that are additive. But the same cannot be said when using counteracting load combinations (see commentary, Section 1604.9). In determining the anchorages required to resist the overturning or uplift effects of wind or seismic loads, resistance is typically provided by the dead load. For example, uplift forces due to wind cause tension to develop in hold-down connections. If the dead load used to resist the wind uplift is overestimated, the result may be an unconservative design. Note that Section 1605.3.2 restricts the dead load used to counteract the effects of overturning and uplift to be the minimum dead load likely to be in place during a design wind event.

As a design guideline, see the unit weights of common construction materials and assemblies in Tables C3-1 and C3-2 of the commentary to ASCE 7. The unit dead loads listed in the tables for assembled elements are usually given in units of pounds per square foot (psf) of surface area (i.e., floor areas, wall areas, ceiling areas, etc.). Unit dead loads for materials used in construction are given in terms of density. The unit weights given in the tables are generally single values, even though a range of weights may actually exist. The average unit weights given are generally suitable for design purposes; however, where there is reason to believe that the actual weights of assembled elements or construction materials may substantially exceed the tabular values, then the situation should be investigated and the highest values used.

**SECTION 1607 LIVE LOADS**

**1607.1 General.** Live loads are those loads defined in Section 1602.1.

Nominal live loads are determined in accordance with Section 1607. The live load requirements for the design of buildings and structures are based on the type of occupancy. Live loads are transient loads that vary with time. Generally, the design live load is that which is believed to be near the maximum transient load for a given occupancy.
TABLE 1607.1. See page 16-20.

The design values of live loads for both uniform and concentrated loads are shown in the table as a function of occupancy. The values given are conservative and include both the sustained and variable portions of the live load. Section 1607.3 directs the designer to utilize the greater live loads produced by the intended occupancy, but not less than the minimum uniformly distributed live loads listed in Table 1607.1. It should also be noted that the “occupancy” category listed is not necessarily group specific. For example, an office building may be classified as Group B, but still contain incidental storage areas. Depending on the type of storage, the areas may warrant storage live loads of either 125 or 250 psf (5.98 or 11.9 kN/m²) to be applied to the space in question.

Table 1607.1 specifies minimum uniform live load in residential attics for three distinct conditions: uninhabitable attics without storage; uninhabitable attics with limited storage; and habitable attics and sleeping areas. Commentary Table 1607.1(1) summarizes the uniform live loads that apply to uninhabitable attics based on the criteria contained in Notes i, j and k to Table 1607.1. The process for determining which load is applicable is summarized in the flow chart in Figure 1607.1.

One noteworthy distinction made by Note k is that any uninhabitable attic that is served by a stairway (other than a pull-down type) must be designed using the load of 30 psf (1.44 kN/m²) that is applicable to habitable attics (see Note k). The implication is that the presence of a stairway is more conducive to using the attic for storage and therefore warrants a greater design live load. This recognizes that the stairway is likely to serve an attic with greater headroom, providing more storage capacity. For an attic that is accessed by other means, such as a framed opening or pull-down stairs, it is necessary to determine whether the attic storage load applies in accordance with the criteria contained in Notes i and j.

Historically, a minimum load of 10 psf (0.48 kN/m²) has been viewed as appropriate where occasional access to the attic is anticipated for maintenance purposes, but significant storage is restricted by physical constraints, such as low clearance or the configuration of truss webs. It provides a minimum degree of structural integrity, allowing for occasional access to an attic space for maintenance purposes. Allowing the application of this load to be independent of other live loads is deemed appropriate, since it would be rare for this load and other maximum live loads to occur at once.

Note m clarifies that a live load reduction is not permitted unless specific exceptions of Section 1607.10 apply. The note appears at each specific use or occupancy in Table 1607.1 where a live load reduction is restricted, which serves to clarify the limitations on live load reduction. References appear in Sections 1607.10.1 and 1607.10.2 to correlate with the note.

<table>
<thead>
<tr>
<th>Description</th>
<th>Uniform Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without storage</td>
<td>10 psf</td>
</tr>
<tr>
<td>With storage</td>
<td>20 psf</td>
</tr>
<tr>
<td>Served by a stairway</td>
<td>30 psf</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 47.88 Pa.

Figure 1607.1
DETERMINATION OF ATTIC LIVE LOAD IN ACCORDANCE WITH TABLE 1607.1 FOOTNOTES i, j and k
### Table 1607.1 Minimum Uniformly Distributed Live Loads, \( L_{up} \) and Minimum Concentrated Live Loads \( L_{con} \)

<table>
<thead>
<tr>
<th>Occupancy or Use</th>
<th>Uniform (psf)</th>
<th>Concentrated (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Apartments (see residential)</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>2. Access floor systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Office use</td>
<td>50</td>
<td>2,000</td>
</tr>
<tr>
<td>Computer use</td>
<td>100</td>
<td>2,000</td>
</tr>
<tr>
<td>3. Armories and drill rooms</td>
<td>150**</td>
<td>—</td>
</tr>
<tr>
<td>4. Assembly areas</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fixed seats (fastened to floor)</td>
<td>60**</td>
<td>—</td>
</tr>
<tr>
<td>Follow spot, projections and control rooms</td>
<td>50</td>
<td>—</td>
</tr>
<tr>
<td>Lobbies</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Movable seats</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Stage floors</td>
<td>150**</td>
<td>—</td>
</tr>
<tr>
<td>Platforms (assembly)</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Other assembly areas</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>5. Balconies and decks*</td>
<td>Same as occupancy served</td>
<td>—</td>
</tr>
<tr>
<td>6. Catwalks</td>
<td>40</td>
<td>300</td>
</tr>
<tr>
<td>7. Cornices</td>
<td>60</td>
<td>—</td>
</tr>
<tr>
<td>8. Corridors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>First floor</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>Other floors</td>
<td>Same as occupancy served except as indicated</td>
<td>—</td>
</tr>
<tr>
<td>9. Dining rooms and restaurants</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>10. Dwellings (see residential)</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>11. Elevator machine room grating</td>
<td>—</td>
<td>300</td>
</tr>
<tr>
<td>(on area of 2 inches by 2 inches)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12. Finish light floor plate construction</td>
<td>—</td>
<td>200</td>
</tr>
<tr>
<td>(on area of 1 inch by 1 inch)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13. Fire escapes</td>
<td>100</td>
<td>40</td>
</tr>
<tr>
<td>On single-family dwellings only</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14. Garages (passenger vehicles only)</td>
<td>40*</td>
<td>Note a</td>
</tr>
<tr>
<td>Trucks and buses</td>
<td></td>
<td>See Section 1607.7</td>
</tr>
<tr>
<td>15. Handrails, guards and grab bars</td>
<td>See Section 1607.8</td>
<td></td>
</tr>
<tr>
<td>16. Helipads</td>
<td>See Section 1607.6</td>
<td></td>
</tr>
<tr>
<td>17. Hospitals</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corridors above first floor</td>
<td>80</td>
<td>1,000</td>
</tr>
<tr>
<td>Operating rooms, laboratories</td>
<td>60</td>
<td>1,000</td>
</tr>
<tr>
<td>Patient rooms</td>
<td>40</td>
<td>1,000</td>
</tr>
<tr>
<td>18. Hotels (see residential)</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>19. Libraries</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corridors above first floor</td>
<td>80</td>
<td>1,000</td>
</tr>
<tr>
<td>Reading rooms</td>
<td>60</td>
<td>1,000</td>
</tr>
<tr>
<td>Stack rooms</td>
<td>150**</td>
<td>1,000</td>
</tr>
<tr>
<td>20. Manufacturing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Heavy</td>
<td>250**</td>
<td>3,000</td>
</tr>
<tr>
<td>Light</td>
<td>125**</td>
<td>2,000</td>
</tr>
<tr>
<td>21. Marquees</td>
<td>75</td>
<td>—</td>
</tr>
<tr>
<td>22. Office buildings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corridors above first floor</td>
<td>90</td>
<td>2,000</td>
</tr>
<tr>
<td>File and computer rooms shall be designed for heavier loads based on anticipated occupancy</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Lobbies and first-floor corridors</td>
<td>100</td>
<td>2,000</td>
</tr>
<tr>
<td>Offices</td>
<td>50</td>
<td>2,000</td>
</tr>
</tbody>
</table>

### Table 1607.1—continued

<table>
<thead>
<tr>
<th>Occupancy or Use</th>
<th>Uniform (psf)</th>
<th>Concentrated (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>23. Penal institutions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cell blocks</td>
<td>40</td>
<td>—</td>
</tr>
<tr>
<td>Corridors</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>24. Recreational uses:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bowling alleys, poolrooms and similar uses</td>
<td>75**</td>
<td>—</td>
</tr>
<tr>
<td>Dance halls and ballrooms</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Gymnasiums</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Reviewing stands, grandstands and bleachers</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>Stadiums and arenas with fixed seats</td>
<td>60**</td>
<td>—</td>
</tr>
<tr>
<td>(Fastened to floor)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25. Residential</td>
<td></td>
<td></td>
</tr>
<tr>
<td>One- and two-family dwellings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uninhabitable attics without storage( \frac{1}{2} )</td>
<td>10</td>
<td>—</td>
</tr>
<tr>
<td>Uninhabitable attics with storage( \frac{1}{2} )</td>
<td>20</td>
<td>—</td>
</tr>
<tr>
<td>Habitable attics and sleeping areas( \frac{3}{4} )</td>
<td>30</td>
<td>—</td>
</tr>
<tr>
<td>All other areas</td>
<td>40</td>
<td>—</td>
</tr>
<tr>
<td>Hotels and multifamily dwellings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Private rooms and corridors serving them</td>
<td>40</td>
<td>—</td>
</tr>
<tr>
<td>Public rooms and corridors serving them</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>26. Roofs</td>
<td></td>
<td>300</td>
</tr>
<tr>
<td>All roof surfaces subject to maintenance workers</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Awnings and canopies:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fabric construction supported by a skeleton structure</td>
<td>5</td>
<td>nonreducible</td>
</tr>
<tr>
<td>All other construction</td>
<td>20</td>
<td>—</td>
</tr>
<tr>
<td>Ordinary flat, pitched, and curved roofs (that are not occupiable)</td>
<td>20</td>
<td>—</td>
</tr>
<tr>
<td>Where primary roof members are exposed to a work floor, at single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Over manufacturing, storage warehouses, and repair garages</td>
<td>2,000</td>
<td>—</td>
</tr>
<tr>
<td>All other primary roof members</td>
<td>300</td>
<td>—</td>
</tr>
<tr>
<td>Occupable roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof gardens</td>
<td>100</td>
<td>—</td>
</tr>
<tr>
<td>Assembly areas</td>
<td>100**</td>
<td>—</td>
</tr>
<tr>
<td>All other similar areas</td>
<td>Note 1</td>
<td>Note 1</td>
</tr>
<tr>
<td>27. Schools</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Classrooms</td>
<td>40</td>
<td>1,000</td>
</tr>
<tr>
<td>Corridors above first floor</td>
<td>80</td>
<td>1,000</td>
</tr>
<tr>
<td>First-floor corridors</td>
<td>100</td>
<td>1,000</td>
</tr>
<tr>
<td>28. Scantles, skylight ribs and accessible ceilings</td>
<td>—</td>
<td>200</td>
</tr>
<tr>
<td>29. Sidewalks, vehicular drive ways and yards, subject to trucking</td>
<td>250**</td>
<td>8,000**</td>
</tr>
</tbody>
</table>

*(continued)*
### TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, \( L_{\text{u}} \), AND MINIMUM CONCENTRATED LIVE LOADS

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30. Stairs and exits</td>
<td></td>
<td></td>
</tr>
<tr>
<td>One- and two-family dwellings</td>
<td>40</td>
<td>300</td>
</tr>
<tr>
<td>All other</td>
<td>100</td>
<td>300</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy</td>
<td>250</td>
</tr>
<tr>
<td>Light</td>
<td>125</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>32. Stores</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Retail</td>
<td></td>
</tr>
<tr>
<td>First floor</td>
<td>100</td>
</tr>
<tr>
<td>Upper floors</td>
<td>75</td>
</tr>
<tr>
<td>Wholesale, all floors</td>
<td>125</td>
</tr>
</tbody>
</table>

| 33. Vehicle barriers | See Section 1807.8.3 |

| 34. Walkways and elevated platforms (other than exitways) | 60 |

| 35. Yards and terraces, pedestrians | 100 |

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm², 1 square foot = 0.0929 m², 1 pound per square foot = 0.0479 kN/m², 1 pound = 0.004448 kN, 1 pound per cubic foot = 16 kg/m³.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
   1. The nominal bookstack unit height shall not exceed 90 inches;
   2. The nominal shelf depth shall not exceed 12 inches for each face; and
   3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

### TABLE 1607.1—continued
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, \( L_{\text{u}} \), AND MINIMUM CONCENTRATED LIVE LOADS

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 lb/ft².

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

### 1607.2 Loads not specified
For occupancies or uses not designated in Table 1607.1, the live load shall be determined in accordance with a method approved by the building official.

Whenever an occupancy or use of a structure cannot be identified with the listing shown in Table 1607.1, then the live load values used for design are required to be determined by the design professional and subject to the approval of the building official. Aside from the obvious intent of this requirement, however, which is to prescribe a minimum design load value, some caution needs to be exercised by the design professional in determining the appropriate design live load value. For example, the table shows that heavy storage areas must be designed for a uniform live load of 250 psf (11.9 kN/m²). This is a minimum value. Storage warehouses or storage areas within manufacturing facilities containing items, such as automobile parts, electrical goods, coiled steel, plumbing supplies and bulk building materials, generally have live loads ranging between 300 and 400 psf (14.4 and 19 kN/m²). Similarly, storage facilities containing dry goods, paints, oil, groceries or liquor often have loadings that range between 200 and 300 psf (9.6 to 14.4 kN/m²). Another example is a heavy manufacturing facility that makes generators for the electric power industry. Some of the production areas in this type of facility require structural floors that support loads of 1,000 psf (47.9 kN/m²) or more, which is about seven times the live load specified in Table 1607.1.

### 1607.3 Uniform live loads
The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall in no
case be less than the minimum uniformly distributed live loads given in Table 1607.1.

- Studies have shown that building live loads consist of a sustained portion based on the day-to-day use of the facilities, and a variable portion created by unusual events, such as remodeling, temporary storage of materials, the extraordinary assemblage of people for an occasional business meeting or social function (i.e., holiday party) and similar events. The sustained portion of the live load will likely vary during the life of a building because of tenant changes, rearrangement of office space and furnishings, changes in the nature of the occupancy (i.e., number of people or type of business), traffic patterns and so on. In light of this variability of loadings that are apt to be imposed on a building, the code provisions simplify the design procedure by expressing the applicable load as either a uniformly distributed live load or a concentrated live load on the floor area. It should be pointed out that this section does not require the concurrent application of uniform live load and concentrated live load. In other words, this section requires that either the uniform load or the concentrated load be applied, so long as the type of load that produces the greater stress in the structural element under consideration is utilized.

1607.4 Concentrated live loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.3 or the concentrated live loads, in pounds (kiloNewtons), given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area of 2\(\frac{1}{2}\) feet by 2\(\frac{1}{2}\) feet (762 mm by 762 mm) and shall be located so as to produce the maximum load effects in the structural members.

- A building or portion thereof is subjected to concentrated floor loads commensurate with the use of the facility. For example, in Group B, a law office may have stacks of books and files that impose large concentrations of loads on the supporting structural elements. An industrial facility may have a tank full of liquid material on a mezzanine that feeds a machine on the floor below. The structural floor of a stockroom may support heavy bins containing metal parts and so on. The exact locations or nature of such concentrated loadings is not usually known at the time of the design of the building. Furthermore, new sources of concentrated loadings will be added during the life of the structure, while some or all of the existing sources will be relocated; therefore, because of the uncertainties of the sources of concentrated loads, as well as their weights and locations, the code provides typical loads to be used in the design of structural floors consistent with the type of use of the facility. The minimum concentrated loads to be used for design are contained in Table 1607.1. Concentrated loads are not required to be applied simultaneously, with the uniform live loads also specified in Table 1607.1. Concentrated loads are to be applied as an independent load condition at the location on the floor that produces the greatest stress in the structural members being designed. The single concentrated load is to be placed at any location on the floor. For example, in an office use area, the floor system is to be designed for either a 2,000-pound (8897 N) concentrated load (unless the anticipated actual concentrated load is higher) applied at any location in the office area, or the 50 psf (2.40 kN/m²) live load specified in Table 1607.1, whichever results in the greater stress in the supporting structural member.

1607.5 Partition loads. In office buildings and in other buildings where partition locations are subject to change, provisions for partition weight shall be made, whether or not partitions are shown on the construction documents; unless the specified live load exceeds 80 psf (3.83 kN/m²). The partition load shall not be less than a uniformly distributed live load of 15 psf (0.72 kN/m²).

- Provision for the weight of partitions must be made in the structural design. The weight of any built-in partitions should be considered a dead load in accordance with the definition in Section 202. Buildings where partitions are readily relocated must include a live load of 15 psf (0.74 kN/m²) if the uniform floor live load is 80 psf (3.83 kN/m²) or less. This partition allowance is included under live loads because of its variable nature.

1607.6 Helipads. Helipads shall be designed for the following live loads:

1. A uniform live load, \(L\), as specified below. This load shall not be reduced.
   
   1.1. 40 psf (1.92 kN/m²) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.
   
   1.2. 60 psf (2.87 kN/m²) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).

2. A single concentrated live load, \(L\), of 3,000 pounds (13.35 kN) applied over an area of 4.5 inches by 4.5 inches (114 mm by 114 mm) and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load is not required to act concurrently with other uniform or concentrated live loads.

3. Two single concentrated live loads, \(L\), 8 feet (2438 mm) apart applied on the landing pad (representing the helicopter’s two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter, and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 inches by 8 inches (203 mm by 203 mm) and are not required to act concurrently with other uniform or concentrated live loads.

Landing areas designed for a design basis helicopter with maximum take-off weight of 3,000 pounds (13.35 kN) shall
be identified with a 3,000 pound (13.34 kN) weight limitation. The landing area weight limitation shall be indicated by the numeral "3" (kips) located in the bottom right corner of the landing area as viewed from the primary approach path. The indication for the landing area weight limitation shall be a minimum 5 feet (1524 mm) in height.

- Detailed requirements for helistops and heliports, including definitions, can be found in Section 412 of the code. For example Section 412.7.1 requires a minimum landing area size of 20 feet by 20 feet (6096 mm by 6096 mm) for helicopters weighing less than 3,500 pounds (15.57 kN). In addition, Section 2007 of the International Fire Code® (IFC®) regulates helistops and heliports. Section 2007.8 of the IFC requires Federal Aviation Administration (FAA) approval of these facilities.

The term "helipad" is used to describe a helicopter landing area, which is the subject of this provision. These structural design requirements establish the minimum live load criteria that are specific to the design of the landing area and the supporting structural elements.

Items 1 and 2 specify the uniform live load based on the threshold weight of 3,000 pounds (13.34 kN). The majority of helicopters used in general aviation have a gross weight of 3,000 pounds (13.34 kN) or less. With weights comparable to those of passenger vehicles and considering the size of the landing area, the equivalent uniform load on these landing areas is actually lower than the minimum uniform live load required for a passenger vehicle parking garage. Thus, a reduced design live load is permitted for the design of such helipads. Since marking the weight limitation of the landing area is standard practice, as well as an FAA recommendation, indicating the weight limitation on the landing area was included as a condition for using the uniform load associated with helicopters weighing up to 3,000 pounds (13.34 kN).

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000 pound (4536 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

- Sections 1607.7.1 through 1607.7.5 give criteria for addressing heavy vehicle loads, including fire trucks and forklifts.

1607.7.1 Loads. Where any structure does not restrict access for vehicles that exceed a 10,000-pound (4536 kg) gross vehicle weight rating, those portions of the structure subject to such loads shall be designed using the vehicular live loads, including consideration of impact and fatigue, in accordance with the codes and specifications required by the jurisdiction having authority for the design and construction of the roadways and bridges in the same location of the structure.

- Where heavy highway-type vehicles have access onto a structure, this section requires the structure to be designed using the same requirements that are applicable to roadways in that jurisdiction. This load-
ing may in fact be the loading from American Association of State Highway and Transportation Officials (AASHTO), or the loading for other elements such as lids of large detention tanks or utility vaults. The registered design professional should consult with the jurisdiction for design loads for these special conditions.

1607.7.2 Fire truck and emergency vehicles. Where a structure or portions of a structure are accessed and loaded by fire department access vehicles and other similar emergency vehicles, the structure shall be designed for the greater of the following loads:

1. The actual operational loads, including outrigger reactions and contact areas of the vehicles as stipulated and approved by the building official; or
2. The live loading specified in Section 1607.7.1.

- This establishes two criteria for addressing heavy vehicle loads due to fire trucks and other similar emergency vehicles.

1607.7.3 Heavy vehicle garages. Garages designed to accommodate vehicles that exceed a 10,000 pound (4536 kg) gross vehicle weight rating, shall be designed using the live loading specified by Section 1607.7.1. Garages for general vehicle use that may have impact and fatigue, but do not accommodate vehicles above the specified gross weight rating, shall be designed using the live loading specified in Section 1607.7.1. The design for impact and fatigue is not required.

Exception: The vehicular live loads and load placement are allowed to be determined using the actual vehicle weights for the vehicles allowed onto the garage floors, provided such loads and placement are based on rational engineering principles and are approved by the building official, but shall not be less than 50 psf (2.9 kN/m²). This live load shall not be reduced.

- This section helps clarify that the passenger vehicle garage loads are not applicable to garages that accommodate heavier vehicles.

1607.7.4 Forklifts and movable equipment. Where a structure is intended to have forklifts or other movable equipment present, the structure shall be designed for the total vehicle or equipment load and the individual wheel loads for the anticipated vehicles as specified by the owner of the facility. These loads shall be posted per Section 1607.7.5.

- Heavy vehicle loads due to forklifts and other movable equipment require that a structure be designed for the total vehicle or equipment load as well as the individual wheel loads. The owner of the facility needs to make the planned usage of such a vehicle clear to the design team. As a precaution these loads must be posted (see Section 1607.7.5).

1607.7.4.1 Impact and fatigue. Impact loads and fatigue loading shall be considered in the design of the supporting structure. For the purposes of design, the vehicle and wheel loads shall be increased by 30 percent to account for impact.

- This section clarifies that consideration of impact and fatigue loading is a design requirement for structures subjected to heavy moving equipment, such as forklifts.
1607.7.5 Posting. The maximum weight of the vehicles allowed into or on a garage or other structure shall be posted by the owner in accordance with Section 106.1.

- As a precaution against overloading a structure, the maximum weight of the vehicles that are anticipated and used in the design should be posted by the owner (see Section 106.1).

1607.8 Loads on handrails, guards, grab bars, seats and vehicle barriers. Handrails, guards, grab bars, accessible seats, accessible benches and vehicle barriers shall be designed and constructed to the structural loading conditions set forth in this section.

- The requirements of this section are intended to provide an adequate degree of structural strength and stability to handrails, guards, grab bars and vehicle barriers.

1607.8.1 Handrails and guards. Handrails and guards shall be designed to resist a linear load of 50 pounds per linear foot (pf) (0.73 kN/m) in accordance with Section 4.5.1 of ASCE 7. Glass handrail assemblies and guards shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.8.1.1 shall be applied.

2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load less than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

- The loading in this section represents the maximum anticipated load on a handrail or guard due to a crowd of people on the adjacent walking surface. The exceptions allow lower loads for handrails in locations that are not typically open to the public. These loads, depicted in Figure 1607.8.1, are permitted to be applied independent of other loads.

1607.8.1.1 Concentrated load. Handrails and guards shall also be designed to resist a concentrated load of 200 pounds (89 kN) in accordance with Section 4.5.1 of ASCE 7.

- The concentrated loading in this section is not to be applied with any other design load; it is a separate load case. The load simulates the maximum anticipated load from a person grabbing or falling into the handrail or guard.

1607.8.1.2 Intermediate rails. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to resist a concentrated load of 50 pounds (0.22 kN) in accordance with Section 4.5.1 of ASCE 7.

- This is a localized design load for the guard members and is not to be applied with any other loads. It is to be applied horizontally at a 90-degree (1.57 rad) angle with the guard members. The number of balusters that would resist this load are those within the 1 square foot (0.093 m²) area in the plane of the guard as shown in Figure 1607.8.1.2.

```
50 LB CONCENTRATED LOAD ON 1 FT x 1 FT SQUARE AT ANY POINT

For SI: 1 pound per square foot = 47.88 Pa.

Figure 1607.8.1.2
COMPONENT DESIGN LOAD
```

```
ANY ANGLE

200 LB CONCENTRATED LOAD

(ALL BUILDINGS)

LOADING CONDITION 1

ANY ANGLE

50 PLF

(NOT APPLICABLE TO ONE- AND TWO-FAMILY DWELLINGS)

LOADING CONDITION 2

For SI: 1 pound = 0.454 kg, 1 pound per foot = 14.59 N/m.

Figure 1607.8.1
HANDRAIL DESIGN LOAD
```
1607.8.2 Grab bars, shower seats and dressing room bench seats. Grab bars, shower seats and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the grab bar or seat so as to produce the maximum load effects.

- These live loads provide for the normal anticipated loads from the use of the grab bars, shower seats and dressing room bench seats. These structural requirements provide consistency with Americans with Disabilities Act (ADA) Accessibility Guidelines (ADAAG).

1607.8.3 Vehicle barriers. Vehicle barriers for passenger vehicles shall be designed to resist a concentrated load of 6,000 pounds (26.70 kN) in accordance with Section 4.5.3 of ASCE 7. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provisions for traffic railings.

- Vehicle barriers provide a passive restraint system in locations where vehicles could fall to a lower level (see definition of "Vehicle barrier," Section 202). Figure 1607.8.3 depicts criteria for the design of passenger car and light truck vehicle barriers. The 6,000-pound (26.70 kN) load considers impact. The load is applied at a height, h, that is representative of vehicle bumper heights. Due to the variety of barrier configurations and anchorage methods, it is necessary to consider any height within the specified size in order to determine the most critical load effects for design of the barrier. For bus and heavy truck vehicle barrier design criteria, a state's Department of Transportation should be contacted (also see Section 1607.7).

1607.9 Impact loads. The live loads specified in Sections 1607.3 through 1607.8 shall be assumed to include adequate allowance for ordinary impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

- In cases where "unusual" live loads occur in a building that impose impact or vibratory forces on structural elements (i.e., elevators, machinery, crane ways, etc.), additional stresses and deflections are imposed on the structural system. Where unusual vibration (dynamic) and impact loads are likely to occur, the code requires that the structural design take these effects into account. Typically, the dynamic effects are approximated through the application of an equivalent static load equal to the dynamic load effects. In most cases, an equivalent static load is sufficient. A dynamic analysis is usually not required.

1607.9.1 Elevators. Members, elements and components subject to dynamic loads from elevators shall be designed for impact loads and deflection limits prescribed by ASME A17.1.

- The static load of an elevator must be increased to account for the effect of the elevator's motion. For example, when an elevator comes to a stop, the load on the elevator's supports is significantly higher than the weight of the elevator and the occupants. This effect varies with the acceleration and deceleration rate of the elevator. This section clarifies that the impact load from elevators applies specifically to members, elements and components subject to dynamic loading from the elevator mechanism and directs the code user to the elevator standard to determine the increases.

1607.9.2 Machinery. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (1) light machinery, shaft- or motor-driven, 20 percent; and (2) reciprocating machinery or power-
driven units, 50 percent. Percentages shall be increased where specified by the manufacturer.

- The specified increases for machinery loads include the vibration of the equipment, which increases the effective load. The load increase for reciprocating machinery versus rotating shaft-driven machinery is to account for the higher vibration.

1607.10 Reduction in uniform live loads. Except for uniform live loads at roofs, all other minimum uniformly distributed live loads, \( I_{uw} \) in Table 1607.1 are permitted to be reduced in accordance with Section 1607.10.1 or 1607.10.2. Uniform live loads at roofs are permitted to be reduced in accordance with Section 1607.12.2.

- Small floor areas are more likely to be subjected to the full uniform load than larger floor areas. Unloaded or lightly loaded areas tend to reduce the total load on the structural members supporting those floors. The specified uniformly distributed live loads from Table 1607.1 are permitted to be reduced, with some notable exceptions or limitations, in recognition that the larger the tributary area of a structural member, the lower the likelihood that the full live load will be realized. The basis for the live load reduction in Sections 1607.10.1 through 1607.10.1.3 is ASCE 7. An alternative method of live load reduction, retained from legacy model codes, is provided in Section 1607.10.2.

1607.10.1 Basic uniform live load reduction. Subject to the limitations of Sections 1607.10.1.1 through 1607.10.1.3 and Table 1607.1, members for which a value of \( K_{ll}A_T \) is 400 square feet (37.16 m²) or more are permitted to be designed for a reduced uniformly distributed live load, \( L \), in accordance with the following equation:

\[
L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{ll}A_T}} \right) \tag{Equation 16-23}
\]

For SLL: \( L = L_o \left( 0.25 + \frac{4.57}{\sqrt{K_{ll}A_T}} \right) \)

where:

- \( L \) = Reduced design live load per square foot (m²) of area supported by the member.
- \( L_o \) = Unreduced design live load per square foot (m²) of area supported by the member (see Table 1607.1).
- \( K_{ll} \) = Live load element factor (see Table 1607.10.1).
- \( A_T \) = Tributary area, in square feet (m²).

\( L \) shall not be less than \( 0.5I_{uw} \) for members supporting one floor and \( L \) shall not be less than \( 0.4I_{uw} \) for members supporting two or more floors.

- This section provides a method of reducing uniform floor live loads that is based on the provisions of ASCE 7. The concept is that where the design live load is governed by the minimum live loads in Table 1607.1, the actual load on a large area of the floor is very likely to be less than the nominal live load in the table. Thus, the allowable reduction increases with the tributary area of the floor that is supported by a structural member; therefore, a girder that supports a large tributary area would be allowed to be designed for somewhat lower uniform live load than a floor beam that supports a smaller floor area.

The following example demonstrates the live load reduction calculation for the conditions shown in Figure 1607.10.1:

Solution:

For interior beam \( K_{ll} = 2 \) (Table 1607.10.1)

\( K, A_T = (2)(750 \text{ sq. ft.}) = 1500 \text{ sq. ft. (139 m}^2 \)\)

Using Equation 16-23

\[
L = 50 \left( 0.25 + \frac{15}{\sqrt{1500}} \right) = 35 \text{psf (1.68 kN/m²)}
\]

0.5 \( L_o = 25 \text{ psf < 32 psf (1.53 kN/m²)} \)

Use reduced live load, \( L = 32 \text{ psf (1.53 kN/m²)} \)

| TABLE 1607.10.1  
| LIVE LOAD ELEMENT FACTOR, \( K_{ll} \) |
|--------------------------|-----------------|
| ELEMENT                  | \( K_{ll} \)     |
| Interior columns         | 4               |
| Exterior columns without cantilever slabs | 4           |
| Edge columns with cantilever slabs | 3               |
| Corner columns with cantilever slabs | 2               |
| Edge beams without cantilever slabs | 2               |
| Interior beams           | 2               |
| All other members not identified above including: Edge beams with cantilever slabs | 1               |
| Cantilever beams         |                 |
| One-way slabs            |                 |
| Two-way slabs            |                 |
| Members without provisions for continuous shear transfer normal to their span | 1               |

- The purpose of this table is to provide tributary area adjustment factors, \( K_{ll} \), for determining live load reductions in Section 1607.10.1. The factor converts the tributary area of the structural member to an "influence area." This "influence area" of a structural member is considered to be the adjacent floor area from which it derives any of its load. These adjustments to the tributary area range from 1 through 4 based on the type of structural element being designed and are meant to reflect the element's ability to share load with adjacent elements.

1607.10.1.1 One-way slabs. The tributary area, \( A_T \), for use in Equation 16-23 for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

- This section limits the tributary area that can be utilized to determine a live load reduction for one-way slabs.
1607.10.1.2 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m²) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than \( L \) as calculated in Section 1607.10.1.

2. For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

The purpose of this section is to prohibit live load reductions where the live loads exceed 100 psf (4.79 kN/m²). Such live loads are typically intended for storage or related purposes. It is more likely that the full live load will be realized at a given floor level for such occupancies. Thus, reduced live loads are not allowed for these conditions except as described in this section.

In Exception 1, the loads on structural members, such as columns and bearing walls that support two or more floors, are allowed to be reduced by 20 percent. Surveys have indicated that it is rare for the total live load on any story to exceed 80 percent of the tabulated uniform live loads. Conservatively, the full load should apply to beams and girders, but a member supporting multiple floors is allowed some live load reduction.

Recognizing that there are circumstances under which live loads exceed 100 psf (4.79 kN/m²) in occupancies other than storage, Exception 2 allows the registered design professional to present a "rational" live load reduction proposal to be applied for members with larger tributary areas (e.g., girders, columns, foundations, etc.). Examples would be mechanical rooms, electrical rooms, process mezzanines in industrial buildings, etc. These types of areas may have very high localized uniform loads under the equipment footprints, for instance, but the live loads to members having larger tributary areas are much less, on average.

1607.10.1.3 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages.

Exception: The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than \( L \) as calculated in Section 1607.10.1.

This section limits the live load reduction for passenger vehicle garages to only those members that support more than two floors. Thus, floor framing members that support only a part of one floor do not warrant a reduction to the live load of 40 psf (1.92 kN/m²) that is specified in Table 1607.1. The rationale for allowing some live load reduction for members supporting multiple floors is similar to that given under Section 1607.10.1.2, Exception 1.

---

![Figure 1607.10.1](image-url)

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m², 1 pound per square foot = 47.88 Pa.

Figure 1607.10.1
LIVE LOAD REDUCTION EXAMPLE
1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m²) except that the design live load for members supporting two or more floors is permitted to be reduced by a maximum of 20 percent.

   **Exception:** For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

2. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.

3. For live loads not exceeding 100 psf (4.79 kN/m²), the design live load for any structural member supporting 150 square feet (13.94 m²) or more is permitted to be reduced in accordance with Equation 16-24.

4. For one-way slabs, the area, A, for use in Equation 16-24 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

   \[ R = 0.08(A - 150) \]  \hspace{1cm} (Equation 16-24)

   For SI: \( R = 0.861(A - 13.94) \)

   Such reduction shall not exceed the smallest of:
   1. 40 percent for horizontal members;
   2. 60 percent for vertical members; or
   3. \( R \) as determined by the following equation.

   \[ R = 23.1(1 + D/L_o) \]  \hspace{1cm} (Equation 16-25)

   where:
   - \( A \) = Area of floor supported by the member, square feet (m²).
   - \( D \) = Dead load per square foot (m²) of area supported.
   - \( L_o \) = Unreduced live load per square foot (m²) of area supported.
   - \( R \) = Reduction in percent.

   **This section includes an alternative floor live load method that is permitted to be used instead of the method indicated in Sections 1607.10.1 through 1607.10.1.3. The basis for this section is the 1997 UBC.**

   Where reductions are permitted, they are allowed at a rate of 0.08 percent per square foot of area in excess of 150 square feet (14 m²). This value cannot exceed 60 percent for vertical members, such as columns or bearing walls, or 40 percent for horizontal members, such as beams or girders. Additionally, the reduction cannot be more than the value determined by Equation 16-25.

Example of an alternate floor live load reduction:

For beam G1 given in Figure 1607.10.1, determine the reduced floor live load in accordance with Section 1607.10.2.

Solution: \( A_f = 750 \text{ square feet (69 m}^2 \) > 150 square feet (14 m²); therefore, a reduction is permitted.

Equation 16-24 \( R = 0.08(750 - 150) = 48% > 40% \)

Equation 16-25

\[ R = (23.1\%)(1 + \frac{45}{50}) = 43.7% > 40% \]

Use the maximum 40 percent reduction allowed for horizontal members.

Use \( L = 50(1-0.4) = 30 \text{ psf (144 kN/m²).} \)

1607.11 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest load effect at each location under consideration. Floor live loads are permitted to be reduced in accordance with Section 1607.10.

For continuous floor members loaded such that the live loads of a building are distributed in some bays and not in others, some of the structural elements will be subjected to greater stresses because of partial loading conditions as compared to full loading on all spans. This code section requires the engineer to consider partial loadings that produce the greatest design forces for any location in the design of continuous floor elements.

For example, Figure 1607.11 shows a continuous multispans girder with partial loading. The Type I loading condition shows that only the alternate spans have uniform live loads, which produces:

- Maximum positive moments at the centers of the loaded spans (A-B, C-D, E-F) and
- Maximum negative moments at the centers of the unloaded spans (B-C, D-E, F-G).

The Type II live load distribution shows two loaded adjacent spans with alternate spans loaded beyond these, which produces:

- Maximum negative moment at Support D and
- Maximum girder shears.

To obtain the maximum total stresses imposed on the girder, the dead load moments and shears must be added to those produced by the partial live loadings.

1607.12 Roof loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads
acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

- In addition to dead and live loads, the roof’s structural system is to be designed and constructed to resist environmental loads caused by wind, snow and earthquakes. According to the definition of “Roof live loads” in Section 202, these are typically an allowance for maintenance of equipment as well as the roof system itself. Other roof live-loads must be considered where appropriate, such as “occupiable roofs” (see Section 1607.12.3) where an occupancy-related live load would be applicable.

1607.12.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m²) in accordance with Section 1607.12.2.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the most unfavorable load effect. See Section 1607.12.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

- For continuous roof construction, where live loads are reduced to less than 20 psf (0.96 kN/m²) as permitted in Section 1607.12.2.1, partial loadings must be included in the design of structural elements to determine the governing loading situation. For example, Figure 1607.11 shows a continuous multispans girder with partial loading. The Type I loading condition shows that only the alternate spans have uniform live loads, which produces:
  - Maximum positive moments at the centers of the loaded spans (A-B, C-D, E-F)
  - Maximum negative moments at the centers of the unloaded spans (B-C, D-E, F-G).

The Type II live load distribution shows two loaded adjacent spans with alternate spans loaded beyond these, which produces:
  - Maximum negative moment at Support D and
  - Maximum girder shears.

To obtain the maximum total stresses imposed on the girder, the dead load moments and shears must be added to those produced by the partial live loadings.

1607.12.2 General. The minimum uniformly distributed live loads of roofs and marquees, \( L_{\text{ur}} \) in Table 1607.1 are permitted to be reduced in accordance with Section 1607.12.2.1.

- The minimum roof live loads typically reflect loads that occur during roof maintenance, construction or repair. In addition to the standard roof live load of 20 psf (0.96 kN/m²), Table 1607.1 includes roof live loads for special purpose roofs and fabric awnings. While this section seems to refer to reducing any of the tabulated uniformly distributed live loads, the actual reduction method provided in Section 1607.12.2.1 is limited to the 20 psf (0.96 kN/m²) live load. This is made evident by the limits on Equation 16-26 of 12 ≤ \( L \) ≤ 20.

1607.12.2.1 Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced uniformly distributed roof live load, \( L_{\text{r}} \), as specified in the following equations or other controlling combinations of loads as specified in Section 1605, whichever produces the greater load effect.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than
specified in the following equations shall not be used unless approved by the building official. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m²).

\[ L_r = L_o R_1 R_2 \]  \hspace{1cm} (Equation 16-26)

where: \( 12 \leq L_r \leq 20 \)

For SI: \( L_r = L_o R_1 R_2 \)

where: \( 0.58 \leq L_r \leq 0.96 \)

\( L_o = \) Unreduced roof live load per square foot (m²) of horizontal projection supported by the member (see Table 1607.1).

\( L_r = \) Reduced roof live load per square foot (m²) of horizontal projection supported by the member.

The reduction factors \( R_1 \) and \( R_2 \) shall be determined as follows:

\( R_1 = 1 \) for \( A_i \leq 200 \) square feet (18.58 m²) \hspace{1cm} (Equation 16-27)

\( R_1 = 1.2 - 0.001 A_i \) for 200 square feet < \( A_i < 600 \) square feet \hspace{1cm} (Equation 16-28)

For SI: \( 1.2 - 0.0114 A_i \) for 18.58 square meters < \( A_i < 55.74 \) square meters

\( R_1 = 0.6 \) for \( A_i \geq 600 \) square feet (55.74 m²) \hspace{1cm} (Equation 16-29)

where:

\( A_i = \) Tributary area (span length multiplied by effective width) in square feet (m²) supported by the member, and

\( R_o = 1 \) for \( F \leq 4 \) \hspace{1cm} (Equation 16-30)

\( R_o = 1.2 - 0.05 F \) for \( 4 < F < 12 \) \hspace{1cm} (Equation 16-31)

\( R_o = 0.6 \) for \( F \geq 12 \) \hspace{1cm} (Equation 16-32)

\( F = \) For a sloped roof, the number of inches of rise per foot (for SI: \( F = 0.12 \times \) slope, with slope expressed as a percentage), or for an arch or dome, the rise-to-span ratio multiplied by 32.

This section provides a formula for the determination of the live load for the design of flat, pitched or curved roofs. The live load from Table 1607.1 that applies is 20 psf (0.96 kN/m²). Reduced roof live loads are based on the roof slope and the tributary area of the member being considered. The portion of the live load reduction based on tributary area does not apply to roof members that support small tributary areas of less than 200 square feet (18.58 m²). The load can be reduced as the tributary area increases but never to less than 12 psf (0.58 kN/m²). For roof slopes between 4:12 and 12:12, live load reductions based on slope apply. Figure 1607.12.2.1 shows the roof

For SI: 1 pound per square foot = 47.88 Pa, 1 square foot = 0.0929 m².

**Figure 1607.12.2.1**
MINIMUM ROOF LIVE LOAD IN ACCORDANCE WITH EQUATION 16-26
live load, $L_o$, determined by Equation 16-26 for increments of roof slope. Since the relationship of variables is linear, intermediate values can be interpolated from the figure.

This section also provides for a lower roof live load for a greenhouse, since it is not likely that loads from maintenance or repair will exceed the specified 12 psf (0.58 kN/m²).

1607.12.3 Occupiable roofs. Areas of roofs that are occupiable, such as roof gardens, or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.10.

- Roofs that are to be occupied during social events incidental to the principal use of the facility are to be designed for a minimum uniform live load of 60 psf (2.87 kN/m²). The promenade deck of a residential penthouse located on the main roof of an apartment building is an example of this type of use. Where roofs are designed to be used as roof gardens or to support large gatherings of people as a function accompanying the educational or assembly uses of a facility, the roofs are required to be designed to a minimum live load of 100 psf (4.79 kN/m²). The minimum live loads specified in Table 1607.1 are not required to be added to the design load requirements for occupiable roofs specified in this section.

1607.12.3.1 Landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m²). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

- Those areas of a roof that are to be landscaped are required to be designed for a minimum uniform live load of 20 psf (0.96 kN/m²) to accommodate the occasional loads associated with the maintenance of plantings. The weight of landscaping materials and saturated soil is to be considered dead load in the design of the roof structure, which is to be combined with the live load (see Section 1605 for applicable load combinations).

1607.12.4 Awnings and canopies. Awnings and canopies shall be designed for uniform live loads as required in Table 1607.1 as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

- Awnings are lightweight frames that are typically covered with fabric materials and are designed to sustain a live load of 5 psf (0.24 kN/m²), as well as the specified snow and wind loads of Sections 1608 and 1609. The live load, snow load and wind load are to be combined according to Section 1605.

1607.13 Crane loads. The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral and longitudinal forces induced by the moving crane.

- This section provides a general description of the crane loads that are required to be included in the design. The supporting structure for the crane is to be designed for a combination of the maximum wheel load, vertical impact and horizontal load as a simultaneous load combination. The typical arrangement for a top-running bridge crane is shown in Figure 1607.13.

1607.13.1 Maximum wheel load. The maximum wheel loads shall be the wheel loads produced by the weight of the bridge, as applicable, plus the sum of the rated capacity and the weight of the trolley with the trolley positioned on its runway at the location where the resulting load effect is maximum.

- The maximum vertical wheel load occurs when the trolley is moved as close as possible to the supporting beams under consideration. This results in the greatest portion of the crane weight, the design weight lifted load and the wheel vertical impact load on the supporting beams.

1607.13.2 Vertical impact force. The maximum wheel loads of the crane shall be increased by the percentages shown below to determine the induced vertical impact or vibration force:

- Monorail cranes (powered) .................. 25 percent
- Cab-operated or remotely operated bridge cranes (powered) .................. 25 percent
- Pendant-operated bridge cranes (powered) .................. 10 percent
- Bridge cranes or monorail cranes with hand-gear bridge, trolley and hoist .................. 0 percent

- A vertical impact force is necessary to account for the impact from the starting and stopping movement of the suspended weight from the crane. Vertical impact is also created by the movement of the crane along the rails.

1607.13.3 Lateral force. The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed with due regard to the lateral stiffness of the runway beam and supporting structure.

- This section is necessary to define the design lateral force on the crane supports. Lateral force at the right angle to the crane rail is caused by the lateral movement of the lifted load and from the frame action of the crane.
1607.13.4 Longitudinal force. The longitudinal force on crane runway beams, except for bridge cranes with hand- geared bridges, shall be calculated as 10 percent of the maximum wheel loads of the crane. The longitudinal force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction parallel to the beam.

- This section is needed to define the longitudinal force on the crane supports, which is caused from the longitudinal motion of the crane with the lifted load.

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m²).

Exception: Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m²).

- The minimum lateral live load is intended to provide sufficient strength and durability of the wall framing and of the finished construction to provide a minimum level of resistance to nominal impact loads that commonly occur in the use of a facility, such as impacts from moving furniture or equipment, as well as to resist heating, ventilating and air-conditioning (HVAC) pressurization.

1607.14.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the following load conditions:

1. A horizontal distributed load of 5 psf (0.24 kN/m²) applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.

2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203 mm) area [50.3 square inches (32.452 mm²)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

- This section provides criteria for fabric partitions (see definition, Section 202) as an alternative to Section 1607.14. The construction of these partitions is unique, which makes it difficult to meet the full requirements of Section 1607.14. Condition 1 requires the partition framing to be capable of resisting a minimum lateral load. Condition 2 approximates the load of a person leaning against the fabric using their hand as the point of contact. This criterion is based on test standards that are used to evaluate the tip-over resistance of office furniture panel systems that are often used to provide open plan offices.

SECTION 1608
SNOW LOADS

1608.1 General. Design snow loads shall be determined in accordance with Chapter 7 of ASCE 7, but the design roof load shall not be less than that determined by Section 1607.

- The determination of the nominal snow load, S, must be in accordance with this section. The intent of this section is that the code requirements are based on the technical requirements in Chapter 7 of ASCE 7. The snow load provisions in ASCE 7 are based on over 40 years of ground snow load data, and include consideration of thermal resistance of the roof structure, a rain-on-snow surcharge, partial loading on...
continuous beam systems and ponding instability from melting snow or rain on snow. The variables that affect the determination of roof snow loads are:

- Ground snow load (pₙ) – see Section 1608.2.
- Importance factor (I) – a factor determined in Section 7.3.3 of ASCE 7, ranging from 0.8 to 1.2, based on the risk category assigned in accordance with Section 1604.5.
- Exposure factor (Cₑ) and Thermal factor (Cₜ) – see Sections 7.3.1 and 7.3.2 of ASCE 7.

In addition to the above, the roof slope affects the snow load determination, as well as other design considerations. Roofs with low slopes are designed for a flat roof snow load, pₙ determined in accordance with Section 7.3 of ASCE 7, using the previously mentioned criteria. Other considerations for low roof slopes include ponding instability and rain-on-snow surcharge loading. For sloped roofs, the sloped roof snow load, pₛ in accordance with Section 7.4 of ASCE 7 applies. The sloped roof snow load essentially modifies the flat roof snow load by the slope factor (Cₑₛ), which varies from 1.0 at low a slope (i.e., no effect) to zero at a slope of 70 degrees (1.36 rad).

The flat roof snow load or sloped roof snow load applied uniformly to the entire roof is referred to as the balanced snow load condition. This loading is always a design consideration. Depending on factors such as the type of roof structural system, the geometry of the roof, etc., the following additional snow loadings may require evaluation:

- Partial loading (see Section 7.5, ASCE 7) is a pattern consisting of balanced snow load and one-half of balanced snow load arranged to produce the maximum effects on the structural member being considered.
- Unbalanced snow loads (see Section 7.6, ASCE 7) reflect an uneven loading pattern, such as can occur on a sawtooth roof [see Figure 1608.1(1)].
- Drifting (see Sections 7.7 and 7.8, ASCE 7) is a concern where adjacent roof surfaces are at different elevations [see Figures 1608.1(2) and (3)] or at projections above the roof level, such as at equipment or parapets.

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs shall be determined in accordance with ASCE 7 or Figure 1608.2 for the contiguous United States and Table 1608.2 for Alaska. Site-specific case studies shall be made in areas designated “CS” in Figure 1608.2. Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be approved. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2-percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions as approved by the building official.

Figure 1608.1(1) BALANCED AND UNBALANCED LOADS FOR A SAWTOOTH ROOF
cates the highest elevation that is appropriate for the use of the associated snow load. Where the elevation limit is exceeded, a site-specific case study is necessary to establish the appropriate ground snow load. In some areas the ground snow load is too variable to allow mapping. These areas are noted as "CS," which indicates a site-specific case study is necessary. Assistance in the determination of an appropriate ground snow load for these areas may be obtained from the U.S. Department of Army Cold Regions Research and Engineering Laboratory (CRREL) in Hanover, New Hampshire.

TABLE 1608.2. See page 16-35.

Since Alaska is not shown on Figure 1608.2, this table provides the ground snow loads, \( P_g \), for Alaskan locations. These values are needed to determine the appropriate roof snow loads for the indicated locations. The roof snow load is to be determined according to Section 7.3 of ASCE 7 for flat roofs and Section 7.4 of ASCE 7 for sloped roofs.

![Figure 1608.1(2) DRIFTING SNOW ON LOW ROOFS AND DECKS](image)

![Figure 1608.1(3) DRIFTING SNOW ONTO ADJACENT LOW STRUCTURE](image)

For SI: 1 foot = 304.8 mm.
FIGURE 1608.2. See page 16-36.

See the commentary to Section 1608.2 for an overview of the snow load map. See the commentary for Section 7.2 of ASCE 7 for a complete description of methodology used in developing the contour lines shown on the figure.

1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 7.11 of ASCE 7.

Susceptible bays of roofs are required to meet the technical provisions of ASCE 7 for consideration of progressive deflection. The term "Susceptible bay" is defined in Section 202 and provides a technical basis for determining which bays of a roof need to be investigated for ponding instability. Section 1611.2 also relies on the determination of which bays are susceptible bays.

![Diagram of ROOF PROJECTION and DRIFT SURCHARGE](image)

Figure 1608.1(4)
SNOW DRIFTING AT ROOF PROJECTIONS

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>POUNDS PER SQUARE FOOT</th>
<th>LOCATION</th>
<th>POUNDS PER SQUARE FOOT</th>
<th>LOCATION</th>
<th>POUNDS PER SQUARE FOOT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adak</td>
<td>30</td>
<td>Galena</td>
<td>60</td>
<td>Petersburg</td>
<td>150</td>
</tr>
<tr>
<td>Anchorage</td>
<td>50</td>
<td>Gulkana</td>
<td>70</td>
<td>St. Paul Islands</td>
<td>40</td>
</tr>
<tr>
<td>Angoon</td>
<td>70</td>
<td>Homer</td>
<td>40</td>
<td>Seward</td>
<td>50</td>
</tr>
<tr>
<td>Barrow</td>
<td>25</td>
<td>Juneau</td>
<td>60</td>
<td>Shemya</td>
<td>25</td>
</tr>
<tr>
<td>Barter Island</td>
<td>35</td>
<td>Kenai</td>
<td>70</td>
<td>Sitka</td>
<td>50</td>
</tr>
<tr>
<td>Bethel</td>
<td>40</td>
<td>Kodiak</td>
<td>30</td>
<td>Talkeetna</td>
<td>120</td>
</tr>
<tr>
<td>Big Delta</td>
<td>50</td>
<td>Kotzebue</td>
<td>60</td>
<td>Unalakleet</td>
<td>50</td>
</tr>
<tr>
<td>Cold Bay</td>
<td>25</td>
<td>McGrath</td>
<td>70</td>
<td>Valdez</td>
<td>160</td>
</tr>
<tr>
<td>Cordova</td>
<td>100</td>
<td>Nenana</td>
<td>80</td>
<td>Whittier</td>
<td>300</td>
</tr>
<tr>
<td>Fairbanks</td>
<td>60</td>
<td>Nome</td>
<td>70</td>
<td>Wrangell</td>
<td>60</td>
</tr>
<tr>
<td>Fort Yukon</td>
<td>60</td>
<td>Palmer</td>
<td>50</td>
<td>Yakutat</td>
<td>150</td>
</tr>
</tbody>
</table>

For SI: 1 pound per square foot = 0.0479 kN/m².
In CS areas, site-specific Case Studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Numbers in parentheses represent the upper elevation limits in feet for the ground snow load values presented below. Site-specific case studies are required to establish ground snow loads at elevations not covered.

To convert lb/sq ft to kN/m², multiply by 0.0479.

To convert feet to meters, multiply by 0.3048.
FIGURE 1608.2—continued
GROUND SNOW LOADS, $p_s$ FOR THE UNITED STATES (psf)
STRUCTURAL DESIGN

SECTION 1609
WIND LOADS


- Over the past decade, new data and research have indicated that the mapped hurricane wind speeds have been overly conservative. Significantly more hurricane data has become available, which in turn allows for improvements in the hurricane simulation model that is used to develop wind speed maps. The new hurricane hazard model yields hurricane wind speeds that are lower than those given in previous editions of ASCE 7 and the code, even though the overall rate of intense storms (as defined by central pressure) produced by the new model is increased compared to those produced by the hurricane simulation model used to develop previous maps. In preparing new maps, it was decided to use multiple ultimate event or strength design maps in conjunction with a wind load factor of 1.0 for strength design (see Section 1605.2). For allowable stress design, the load factor has been reduced from 1.0 to 0.6 (see Section 1605.3). Several factors that are important to an accurate wind load standard led to this decision:

1. An ultimate event or strength design wind speed map makes the overall approach consistent with that used in seismic design in that they both map ultimate events and use a load factor of 1.0 for strength design.

2. Utilizing different maps for the different risk categories eliminates the problems associated with using "importance factors" that vary with category. The difference in the importance factors in hurricane-prone and nonhurricane-prone regions for Risk Category I structures, which prompted many questions, is gone.

3. The use of multiple maps eliminates the confusion associated with the recurrence interval associated with the previous wind speed map—the map was not a uniform 50-year return period map. This created a situation where the level of safety that was provided within the overall design was not consistent along the hurricane coast.

Utilizing the new wind speed maps and integrating their use into the code necessitated the introduction of the terms "$V_{ult}$" and "$V_{nom}$" to be associated with the "ultimate" design wind speed and the "nominal" design wind speed, respectively. Because of the number of different provisions that use the wind speed map to "trigger" different requirements, it was necessary to provide a conversion methodology in Section 1609.3.1 so that those provisions were not affected. The terms "ultimate design wind speed" and "nominal design wind speed" have been incorporated to clarify the different levels of wind speed.

Prior to 1998, earlier editions of ASCE 7 as well as the legacy model codes incorporated fastest-mile wind speed maps. "Fastest mile" is defined as the average speed of one mile of air that passes a specific reference point. It is important to recognize the differences in averaging times between fastest-mile and the 3-second-gust maps. The averaging time for a 90-mph fastest-mile wind speed is $(t = 3,600/V) 40$ seconds. Obviously, due to greater averaging time, for a given location the fastest-mile wind speed will be less than the 3-second-gust wind speed. The fact that the wind speed values are higher does not necessarily indicate higher wind loads. Buildings and structures resist wind loads, not wind speeds. Wind speed, although a significant contributor, is only one of several variables and factors that affect wind forces. Wind loads are affected by atmosphere and aerodynamics. Other elements that affect actual wind forces as wind flows across a bluff body include shape factors ($C_s$), gust effect factors ($G$) and the velocity pressure that is a function of wind speed, exposure and topography, among others.

The change from the fastest-mile wind speed map to a 3-second-gust map in the 2000 code (based on the 1998 edition of ASCE 7) was necessary for the following reasons. First, weather stations across the United States no longer collect fastest-mile wind speed data. Additionally, the perception of the general public will be more favorable where the code wind speeds are higher, although the design wind pressures were not changed significantly. The map includes a more complete analysis of hurricane wind speeds than previous maps, since more data was available for sites away from the coast. In western states, the 85- to 90-mph contour boundary follows along the Washington, Oregon and California eastern state lines. This is because inland wind data was such that there was no statistical basis to place them elsewhere. The reference to the 50-year MRI used in previous maps was removed, reflecting the fact that the MRI is greater than 50 years along the hurricane coastline. However, nonhurricane wind speeds are based on a 50-year MRI, as Section 1609.3 specifically requires when estimating basic wind speeds from local climatic data.

1609.1 Applications. Buildings, structures and parts thereof shall be designed to withstand the minimum wind loads prescribed herein. Decreases in wind loads shall not be made for the effect of shielding by other structures.

- The determination of the wind load, $W$, must be in accordance with Section 1609. The intent of this section is to provide minimum criteria for the design and construction of buildings and other structures to resist wind loads. These regulations serve to reduce the potential for damage to property caused by windstorms and to provide an acceptable level of protection to building occupants. The objective also includes the prevention of damage to adjacent properties because of the possible detachment of major building components (e.g., walls, roofs, etc.), structural collapse or flying debris and for the safety of people in the immediate vicinity.
The criteria for wind design given in this section of the code generally reflect the wind load provisions of ASCE 7. For a better understanding of the wind load provisions it is important to have a fundamental knowledge of the effects of high-velocity wind forces on buildings and other structures. Wind/structure interactions can be characterized as follows:

When wind encounters a stationary object, such as a building, the airflow changes direction and produces several effects on the building that are illustrated in Figure 1609.1(1). Exterior walls and other vertical surfaces facing the wind (windward side) and perpendicular to its path are subjected to inward (positive) pressures; however, wind does not stop on contact with a facing surface, but flows around and over the building surfaces. This airflow does not instantaneously change directions at surface discontinuities, such as corners of walls or eaves, or over ridges and roof corners. Instead it separates from the downwind surfaces due to high turbulence and localized pressures, resulting in outward (negative) pressures. This phenomena produces suction or outward pressures (negative) on the sidewalls, leeward wall and, depending on geometry, the roof.

Figure 1609.1(1) shows a flat roof and the resulting negative pressures caused by external wind; however, pressures may differ on sloping roofs in the direction perpendicular to the ridge. Roof surfaces (on the windward side) with shallow slopes are generally subjected to outward (negative) pressures—the same as flat roofs. Moderately sloping roofs [about 30 degrees (0.5235 rad)] may be subjected to either inward (positive) pressures, outward (negative) pressures or both—negative pressure in the lower part of the roof and positive pressure in the area of the ridge; however, the code does not require consideration of this scenario. High-sloping roofs (windward side) respond similar to walls and sustain positive wind pressures. The leeward side of a sloped roof is subjected to negative pressure, regardless of the angle. When the wind acts parallel to the ridge, the pressures on a sloped roof are similar to the pressures on a flat roof, meaning the roof is subject to negative pressures.

Openings in the building envelope can impact the magnitude of wind pressure on a structure by increasing the internal pressures that, in turn, will affect the net pressures on the structure. Figure 1609.1(2) illustrates the effects of openings in a building’s exterior walls. An opening in the leeward wall causes negative pressure on the interior, increasing the total load on the windward wall. Openings on the windward wall cause positive internal pressure against all the walls from the interior of the building. As the figure shows, the result is an increase in the total load on the leeward wall. In extreme cases, fail-
ures resulting from this type of wind flow appear as if the building has exploded.

The definition of "Openings" is worth noting, since in the nonstructural code provisions the term "openings" typically refers to doors, ducts or windows. The definition of "Openings" that is relevant to wind load is in Section 26.2 of ASCE 7. They are all described as "apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as 'open' during design wind loads." Such openings are then considered in classifying the building as enclosed, partially enclosed or open. The wind load considerations will differ based on that classification.

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, \(V_{aw}\), and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of ASI S230.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 26.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, \(V_{aw}\), and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, \(V_{aw}'\), when the provisions of the standards referenced in Exceptions 1 through 5 are used.

The intent of Section 1609 is to require that buildings and structures be designed and constructed to resist the wind loads quantified in Chapters 26 through 30 of ASCE 7.

There are six exceptions to using the provisions of ASCE 7 for the determination of wind loads.

Exception 1 provides for the use of ICC 600 for Group R-2 and R-3 buildings where they are located within Exposure B or C as defined in Section 1609.4 and not sited on the upper half of an isolated hill, escarpment or ridge with the characteristics described in Section 1609.1.1.1. ICC 600 has prescriptive construction requirements and required load capacity tables that replace the requirement for structural analysis, which is intended to provide improved design construction details to achieve greater structural performance for single- and multiple-family dwellings in a high-wind event (see ICC 600 for other detailed application limitations).

Exception 2 provides for the use of the AF&PA Wood Frame Construction Manual for One- and Two-family Dwellings where the building is sited within Exposure B or C as defined in Section 1609.4 and not on the upper half of an isolated hill, escarpment or ridge with the characteristics described in Section 1609.1.1.1. The AF&PA Wood Frame Construction Manual for One- and Two-family Dwellings has prescriptive construction requirements and required load-resistance tables that replace the requirement for structural analysis. The tabulated engineered and prescriptive design provisions apply to one- and two-family wood frame dwellings where the fastest-mile basic wind speed is between 90 and 120 mph. See Table 1609.3.1 for conversion between the 3-second gust and fastest-mile wind speeds (see the AF&PA Wood Frame Construction Manual for One- and Two-family Dwellings, Chapter 1, for other detailed application limitations). Similarly, Exception 3 allows the cold-formed steel prescriptive standard for one- and two-family dwellings.

Exceptions 4 and 5 simply refer to national standards dealing specifically with the design of flag poles and telecommunication towers.

Exception 5 allows the use of TIA 222-G for antennas and their supporting structures. This standard provides a simplification of the topographic wind speed-up effect. The TIA 222-G standard allows a designer to use the full topographic wind speed-up method of ASCE 7 in order to avoid the conservatism of the simplified method. A code modification makes sure that the simplified method is safe in all cases.

TIA 222-G accounts for the worst-case wind speed-up at the crest for a steep slope, but overlooks the fact that lesser sloped escarpments create pressure increases that cannot be safely ignored beyond the "steep slope" influence. As written, the standard stops considering the topographic wind speed-up effect at "8 x height" from the crest. At this distance, a shallow slope can still increase wind pressure by more than 20 percent.

The modification changes "8 x height" in the standard to "16 x height." The intent is that the "Topographic category 2" definition in TIA 222-G should be applied as follows:

Category 2. Structures located at or near the crest of an escarpment. Wind speed-up shall be considered to occur in all directions. Structures located vertically on the lower half of an escarpment or horizontally beyond 16 times the height of the escarpment from its crest, shall be permitted to be considered as Topographic Category 1.
Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

FIGURE 1609A
ULTIMATE DESIGN WIND SPEEDS, $V_{GL,P}$ FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES
FIGURE 1609B
ULTIMATE DESIGN WIND SPEEDS, $V_{ult}$ FOR RISK CATEGORY III AND IV BUILDINGS AND OTHER STRUCTURES

**TEXT:**

Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.

Linear interpolation between contours is permitted.

Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).
Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

FIGURE 1609C
ULTIMATE DESIGN WIND SPEEDS, $V_{w,i}$ FOR RISK CATEGORY I BUILDINGS AND OTHER STRUCTURES
1609.1.1.1 Applicability. The provisions of ICC 600 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ICC 600, AF&PA WFCM and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;
2. The maximum average slope of the hill exceeds 10 percent; and
3. The hill, ridge or escarpment is unobstructed upwind by other such topographic features for a distance from the high point of 50 times the height of the hill or 1 mile (1.61 km), whichever is greater.

This section places limitations on the use of ICC 600 and the AF&PA Wood Frame Construction Manual for One- and Two-family Dwellings. Neither of these standards accounts for the effect of isolated hills, ridges or escarpments. As illustrated in Figure 1609.1.1.1, the wind speed increases when a mass of air passes over those terrain features. This phenomenon is referred to as the "wind speed-up" effect and requires the use of ASCE 7.

1609.1.2 Protection of openings. In wind-borne debris regions, glazing in buildings shall be impact resistant or protected with an impact-resistant covering meeting the requirements of an approved impact-resistant standard or ASTM E 1996 and ASTM E 1886 referenced herein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E 1996.

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where $V_{ud}$ determined in accordance with Section 1609.3.1 does not exceed 140 mph (63 m/s).

2. Glazing in Risk Category I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.

3. Glazing in Risk Category II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

The purpose of this section is to address risks associated with wind-borne debris in high-wind areas. See the definitions of "Wind-borne debris region" and "Hurricane-prone regions" in Section 1609.2 for an explanation of where these provisions apply. This section requires protection of glazed openings in buildings in wind-borne debris regions. This can be provided in the form of a protective assembly that is impact tested or by using impact-resistant glazing. During a hurricane, buildings are impacted from
wind-borne debris due to high-velocity winds. This debris can impact the glazing, causing breakage and creating an opening within the building envelope. The presence of openings in the building envelope can have a significant effect on the magnitude of the total wind pressure required to be resisted by each structural element of a building. Depending on the location of these openings with respect to wind direction and the amount of background porosity, external and internal pressures may act in the same direction to produce higher forces on some walls and the roof. An example of this is shown in Figure 1609.1.2. In this scenario, as the wind flows over the building, pressures are developed on the external surface, as shown. Introduction of an opening in the windward wall causes the wind to rush into the building, exerting internal pressures (positive) against all interior surfaces. This type of opening has the net effect of producing potentially high internal pressures that will act in the same direction as the external pressures on the roof, side and leeward walls. Considering the high probability of wind-borne debris during a hurricane and the effect of an unintended opening in the building envelope, the code requires glazing in designated regions to be protected.

Where wind-borne protection is provided, the section specifies two types of tests to demonstrate adequate resistance: the large missile test to simulate large debris up to 30 feet (9144 mm) above grade and the small missile test to simulate smaller debris up to 60 feet (18 288 mm) above grade, both of which are common during very high winds. An example of small debris is gravel from the surrounding area that becomes airborne.

Impact-resistant coverings or glazing must meet the test requirements of an approved impact standard or ASTM E 1886 and ASTM E 1996. Other impact standards the building official may consider are SBCCI SSTD 12 or Florida Building Code Test Protocol TAS 201, TAS 202 and TAS 203. These standards specify similar-type testing with a large missile test (2 by 4), small missile test (2 gram balls) and cyclic pressure loading test. ASTM E 1886 and ASTM E 1996 work together with the standard test method (E 1886) and the test specification (E 1996), including scoping, technical requirements and pass/fail criteria.

Exception 1 permits the use of 7/16-inch (11.1 mm) wood structural panels with maximum spans of 8 feet (2438 mm) as an impact-resistant covering for one- and two-story buildings. Panel attachments have to be designed to resist the component and cladding pressure from ASCE 7 or attached in accordance with Table 1609.1.2. This protective system has been tested and meets the requirements of SBCCI SSTD 12. The intent is that precut panel coverings and attachment hardware are provided on site.

Exception 2 exempts low-hazard (Risk Category I) buildings from this requirement for protecting openings against wind-borne debris.

Exception 3 exempts openings in buildings that are classified in Risk Category II, III or IV where the opening locations meet the stated conditions.

**TABLE 1609.1.2.** See page 16-46.

- This table provides the connections for the wood structural panel impact-resistant covering that is described in the exception to Section 1609.1.2. The table lists the spacing of the screws around the perimeter of the panel for the indicated panel spans. Note that Table 1609.1.2 is only applicable to buildings with a mean roof height of 33 feet (10 058 mm) or less and located where the basic wind speed is 130 mph or less.

---

![Figure 1609.1.2](image)
TABLE 1609.1.2 WIND-BORNE DEBRIS PROTECTION FASTENING SCHEDULE FOR WOOD STRUCTURAL PANELS

<table>
<thead>
<tr>
<th>FASTENER TYPE</th>
<th>FASTENER SPACING (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Panel Span ≤ 4 feet</td>
</tr>
<tr>
<td>No. 8 wood-screw-based anchor with 2-inch embedment length</td>
<td>16</td>
</tr>
<tr>
<td>No. 10 wood-screw-based anchor with 2-inch embedment length</td>
<td>16</td>
</tr>
<tr>
<td>3/4-inch diameter lag-screw-based anchor with 2-inch embedment length</td>
<td>16</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448 N, 1 mile per hour = 0.447 m/s.

a. This table is based on 140 mph wind speeds and a 45-foot mean roof height.
b. Fasteners shall be installed at opposing ends of the wood structural panel.
c. Fasteners shall be located a minimum of 1 inch from the edge of the panel.
d. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. Fasteners shall be located a minimum of 21/8 inches from the edge of concrete block or concrete.
e. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1,500 pounds.

1609.1.2.1 Louvers. Louvers protecting intake and exhaust ventilation ducts not assumed to be open that are located within 30 feet (9144 mm) of grade shall meet the requirements of AMCA 54.

This section provides direction for impact testing of louvers that cover intake or exhaust duct openings in wind-borne debris regions. Louvers often have blades or slats affixed to and covering an opening in the exterior envelope, making them similar to certain types of porous shutters used to protect glazed openings. When a louver in an exterior wall is damaged by wind-borne debris during a high-wind event, the air-leakage-rated damper inside the ventilation duct may also be exposed to damage.

The scope of ASTM E 1996 covers impact testing of exterior building features, such as windows, glazed curtain walls, doors and storm shutters, in buildings located in geographic regions that are prone to hurricanes, simulating impact by both large and small missiles. For glazed openings and nonporous shutters that protect the fenestration assembly, the impact testing is followed by a cyclic loading test. There is no specific provision in the standard for testing louvers that cover ventilation openings, either for impact or air pressure cycling. Nevertheless, in the absence of an appropriate specification and test method for louvers, some jurisdictions have based their product approvals on the performance criteria of the large missile test of ASTM E 1886 and ASTM E 1996. The Air Movement Control Association (AMCA) has developed a standard specification for louvers that provides a uniform set of guidelines and a consistent basis for evaluating the ability of the louver to maintain its integrity during the large missile test of ASTM E 1886 and ASTM E 1996.

1609.1.2.2 Application of ASTM E 1996. The text of Section 6.2.2 of ASTM E 1996 shall be substituted as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the design wind speed, Vwind, as follows:

6.2.2.1 Wind Zone 1—130 mph ≤ ultimate design wind speed, Vwind ≤ 140 mph.

6.2.2.2 Wind Zone 2—140 mph ≤ ultimate design wind speed, Vwind ≤ 150 mph at greater than one mile (1.6 km) from the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.3 Wind Zone 3—150 mph (58 m/s) ≤ ultimate design wind speed, Vwind ≤ 160 mph (63 m/s), or 140 mph (54 m/s) ≤ ultimate design wind speed, Vwind ≤ 160 mph (63 m/s) and within one mile (1.6 km) of the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.4 Wind Zone 4—ultimate design wind speed, Vwind >160 mph (63 m/s).

The purpose of this section is to correlate the wind zones of ASTM E 1996 with the new wind speed maps. It is needed to delineate the wind zones because ASTM E 1996 does not use Vwind.

1609.1.2.3 Garage doors. Garage door glazed opening protection for wind-borne debris shall meet the requirements of an approved impact-resisting standard or ANSI/DASMA 115.

This provision references a standard for the wind-borne debris resistance testing of glazing installed in garage doors. Because ASTM E 1886 and ASTM E 1996 require interpretation regarding their application to garage doors, DASMA 115 is the primary standard referenced for this purpose.

1609.2 Definitions. For the purposes of Section 1609 and as used elsewhere in this code, the following terms are defined in Chapter 2.

Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with wind loads. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

HURRICANE-PRONE REGIONS.

WIND-BORNE DEBRIS REGION.

WIND SPEED, Vwind

WIND SPEED, Vwind

1609.3 Basic wind speed. The ultimate design wind speed, Vwind, in mph, for the determination of the wind loads shall be
The terms "ultimate design wind speed" and "nominal design wind speed" have been incorporated in numerous locations to help the code user distinguish between them. For example, in a case where the code previously imposed requirements where the basic wind speed exceeds 100 mph (45 m/s), it now imposes the requirements where V_{asd} exceeds 100 mph (45 m/s). A nominal design speed, V_{nom} equal to 100 mph (45 m/s) corresponds to an ultimate design wind speed, V_{tail} equal to 129 mph (58 m/s). The conversion between the two is accomplished using Equation 16-32 or Table 1609.3.1. This conversion equation is the result of the wind load being proportional to the square of the velocity pressure and the ASD wind load being 0.6 times the strength level wind load.

It should be noted that the term "basic wind speed" remains in ASCE 7, but it corresponds to the "ultimate design wind speed" in the code. For a comparison of ASCE 7-93 fastest-mile wind speeds and ASCE 7-05 3-second gust (ASD) wind speeds to ASCE 7-10 3-second gust wind speeds, refer to Table C26.5-6 in the ASCE 7-10.

**TABLE 1609.3.1.** See below.

- The table converts the mapped ultimate wind speed to nominal design wind speed for use where necessary, such as in the standards that are referenced in Section 1609.1.1.

**1609.4 Exposure category.** For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.

- The concept of exposure categories provides a means of accounting for the relative roughness of the boundary layer. The earth's surface exerts a horizontal drag force on wind due to ground obstructions that retard the flow of air close to the ground. The reduction in the flow of air is a function of height above ground and terrain roughness. Wind speeds increase with height above ground, and the relationship between height above ground and wind speed is exponential. The rate of increase in wind speeds with height is a function of the terrain features. The rougher the terrain (such as large city centers), the shallower the slope of the wind speed profile. The

<table>
<thead>
<tr>
<th>V_{tail}</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
<th>140</th>
<th>150</th>
<th>160</th>
<th>170</th>
<th>180</th>
<th>190</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_{nom}</td>
<td>78</td>
<td>85</td>
<td>93</td>
<td>101</td>
<td>108</td>
<td>116</td>
<td>124</td>
<td>132</td>
<td>139</td>
<td>147</td>
<td>155</td>
</tr>
</tbody>
</table>

For SI: 1 mile per hour = 0.44 m/s.

- Linear interpolation is permitted.
- V_{nom} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.
- V_{tail} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C.

---

**TABLE 1609.3.1**

WIND SPEED CONVERSIONS

<table>
<thead>
<tr>
<th>V_{tail}</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
<th>140</th>
<th>150</th>
<th>160</th>
<th>170</th>
<th>180</th>
<th>190</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_{nom}</td>
<td>78</td>
<td>85</td>
<td>93</td>
<td>101</td>
<td>108</td>
<td>116</td>
<td>124</td>
<td>132</td>
<td>139</td>
<td>147</td>
<td>155</td>
</tr>
</tbody>
</table>

For SI: 1 mile per hour = 0.44 m/s.

- Linear interpolation is permitted.
- V_{nom} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.
- V_{tail} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C.
smoother the terrain (open water), the steeper the slope of the wind speed profile.

The definitions of exposure and roughness categories are used to account for this roughness in the boundary layer and are intended to provide an adequate assessment of surface roughness for most situations. Exposure B is considered the roughest boundary layer condition. Exposure D is considered the smoothest boundary layer condition. Accordingly, calculated wind loads are less for Exposure B, which has more surface obstructions, as compared to Exposure D, with all other variables the same (see ASCE 7 commentary for guidance on performing a more detailed analysis of surface roughness).

1609.4.1 Wind directions and sectors. For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees (0.79 rad) either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 1609.4.2 and 1609.4.3 and the exposure resulting in the highest wind loads shall be used to represent winds from that direction.

- An exposure category must be determined for each direction in which wind loading is being considered. This section describes the method for doing so, and requires use of the more restrictive exposure category where the two sectors in a given wind direction would be classified in different exposure categories.

1609.4.2 Surface roughness categories. A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

Surface Roughness B. Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, and grasslands.

Surface Roughness D. Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats and unbroken ice.

- This section defines three surface roughness categories that are used in evaluating each sector and, subsequently, in determining an exposure category. These surface roughnesses were previously included in the corresponding exposure category definition. The required upwind distance that must be considered varies according to the exposure category definition in Section 1609.4.3.

A significant philosophical change in determination of exposure categories occurred with the inclusion of shorelines in hurricane-prone regions in the definition of Exposure Category C (Surface Roughness C). Exposure Category D (Surface Roughness D) had been used for wind flowing over open water until further research determined that wave action at the water’s surface in a hurricane, due to the intensity of the turbulence, produced substantial surface obstructions and friction that reduces the wind profile values to be more in line with Surface Roughness C as opposed to Surface Roughness D. Surface Roughness D would still apply to inland waterways and shorelines that are not in the hurricane-prone regions, such as coastal California, Oregon, Washington and Alaska.

1609.4.3 Exposure categories. An exposure category shall be determined in accordance with the following:

Exposure B. For buildings with a mean roof height of less than or equal to 30 feet (9144 mm), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance of at least 1,500 feet (457 m). For buildings with a mean roof height greater than 30 feet (9144 mm), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.

Exposure C. Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 feet (183 m) or 20 times the building height, whichever is greater, from an exposure D condition as defined in the previous sentence.

- This section defines the three exposure categories that are used to determine various wind load parameters. Exposure categories are used along with basic wind speed elsewhere in the code as a threshold for wind requirements, such as establishing the need for structural observation in Section 1704.5. Exposure B is the most common type of exposure category in the country. A study by the National Association of Home Builders (NAHB) indicated that perhaps up to 80 percent of all buildings were located in Exposure B.

1609.5 Roof systems. Roof systems shall be designed and constructed in accordance with Sections 1609.5.1 through 1609.5.3, as applicable.

- This section clarifies the design wind loads that are applied to roof decks and roof coverings.
1609.5.1 Roof deck. The roof deck shall be designed to withstand the wind pressures determined in accordance with ASCE 7.

This section specifies the wind load criteria for the roof deck. The roof deck is a structural component of the building and must resist the applicable wind pressures of ASCE 7. This section is referenced by Section 1609.5.2 as the criteria for the wind design for roof coverings that are relatively impermeable.

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

**Exception:** Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind-resistance requirements of Section 1507.2.7.1.

This section establishes the wind design criteria for roof coverings. The exception references the use of Section 1609.5.3 for air-permeable rigid tile roof coverings. If the roof deck is relatively impermeable, wind pressures will act through it to the building frame system. The roof covering may or may not be subjected to the same wind pressures as the roof deck. If the roof covering is also relatively impermeable and fastened to the roof deck, the two components will react to and resist the same wind pressures. If the roof covering is not impermeable, the wind pressures will be able to develop on both the top of, and underneath, the roof covering. This ‘venting action’ will negate some wind pressure on the roof covering.

1609.5.3 Rigid tile. Wind loads on rigid tile roof coverings shall be determined in accordance with the following equation:

\[
M_a = q_w C_w b L_a [1.0 - G_{CP}]
\]

(Equation 16-34)

For SI: \[
M_a = \frac{L(q_w C_w b) L_a [1.0 - G_{CP}]}{1,000}
\]

where:

- \(b\) = Exposed width, feet (mm) of the roof tile.
- \(C_w\) = Lift coefficient. The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined by test in accordance with Section 1711.2.
- \(G_{CP}\) = Roof pressure coefficient for each applicable roof zone determined from Chapter 30 of ASCE 7. Roof coefficients shall not be adjusted for internal pressure.
- \(L\) = Length, feet (mm) of the roof tile.
- \(L_a\) = Moment arm, feet (mm) from the axis of rotation to the point of uplift on the roof tile. The point of uplift shall be taken at 0.76L from the head of the tile and the middle of the exposed width. For roof tiles with nails or screws (with or without a tail clip), the axis of rotation shall be taken as the head of the tile for direct deck application or as the top edge of the batten for battened applications. For roof tiles fastened only by a nail or screw along the side of the tile, the axis of rotation shall be determined by testing. For roof tiles installed with battens and fastened only by a clip near the tail of the tile, the moment arm shall be determined about the top edge of the batten with consideration given for the point of rotation of the tiles based on straight bond or broken bond and the tile profile.

\[
M_a = \text{Aerodynamic uplift moment, feet-pounds (N-mm)}
\]

acting to raise the tail of the tile.

\[
q_w = \text{Wind velocity pressure, psf (kN/m²) determined from Section 27.3.2 of ASCE 7.}
\]

Concrete and clay roof tiles complying with the following limitations shall be designed to withstand the aerodynamic uplift moment as determined by this section.

1. The roof tiles shall be either loose laid on battens, mechanically fastened, mortar set or adhesive set.
2. The roof tiles shall be installed on solid sheathing which has been designed as components and cladding.
3. An underlayment shall be installed in accordance with Chapter 15.
4. The tile shall be single lapped interlocking with a minimum head lap of not less than 2 inches (51 mm).
5. The length of the tile shall be between 1.0 and 1.75 feet (305 mm and 533 mm).
6. The exposed width of the tile shall be between 0.67 and 1.25 feet (204 mm and 381 mm).
7. The maximum thickness of the tail of the tile shall not exceed 1.3 inches (33 mm).
8. Roof tiles using mortar set or adhesive set systems shall have at least two-thirds of the tile’s area free of mortar or adhesive contact.

This section includes the wind design method for clay or concrete rigid tile roofs. The method consists of the calculation of the aerodynamic uplift moment from the wind that acts to raise the end of the tile. This section includes the characteristics and the type of installation of the concrete or clay roof tile that are required for the use of the design method.

In certain types of installations, the roof covering is not exposed to the same wind loads as the roof deck. Concrete and clay roof tiles are typical of this type of installation and are not subject to wind loads that would be obtained from current wind-loading criteria. This is due to the gaps at tile joints allowing some equalization of pressure between the inner and outer face of the tiles, leading to reduced loads. A procedure has been developed through research for determining the uplift moment on loose-laid and mechanically fastened roof tiles when laid over sheathing with an underlayment. The procedure is
STRUCTURAL DESIGN

based on practical measurements on real tiles to determine the effect of air being able to penetrate the roof covering.

1609.6 Alternate all-heights method. The alternate wind design provisions in this section are simplifications of the ASCE 7 Directional Procedure.

❖ In response to concerns from design engineers on the complexity of wind design procedures, member organizations of the National Council of Standard Engineering Associations (NCSEA) assembled this alternative method for determining wind loads. The procedure has been developed to give results equal to or more conservative than the Directional Procedure that is found in ASCE 7. The intention is to reduce the effort required in determining wind forces for the main wind-force-resisting system, as well as for components and cladding (C & C).

1609.6.1 Scope. As an alternative to ASCE 7 Chapters 27 and 30, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures that are regularly shaped, which meet all of the following conditions:

1. The building or other structure is less than or equal to 75 feet (22 860 mm) in height with a height-to-least-width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.

2. The building or other structure is not sensitive to dynamic effects.

3. The building or other structure is not located on a site where the channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 26.2, where wind loads are only transmitted to the main wind-force-resisting system (MWFRS) at the diaphragms.

5. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, roofs with slopes greater than 45 degrees (0.79 rad), solid free-standing walls and solid signs, and rooftop equipment, apply ASCE 7 provisions.

❖ While ASCE 7 already includes a simplified procedure, it necessarily includes numerous limitations. Similarly, this alternative design procedure is limited, though it was developed to apply to a broader range of buildings.

Note that Item 1 limits application to buildings with a frequency of at least 1 hertz. In other words, this can only be used for rigid structures (see definition of "Rigid buildings and other structures" in Section 26.2 of ASCE 7). Item 1 also allows any building up to 75 feet (22 860 mm) in height that has a height-to-least-width ratio of 4 or less to qualify without calculating the frequency. Buildings must also be regularly shaped, simple diaphragm buildings with envelopes classified as either enclosed or partially enclosed. Item 5 provides a partial list of structures that must be analyzed using the ASCE 7 provisions.

1609.6.1.1 Modifications. The following modifications shall be made to certain subsections in ASCE 7: in Section 1609.6.2, symbols and notations that are specific to this section are used in conjunction with the symbols and notations in ASCE 7 Section 26.3.

❖ These alternative provisions are an adaptation of the ASCE 7 analytical procedure. In providing the intended simplifications, this section alerts the code user that these are modifications to the ASCE 7 procedure.

1609.6.2 Symbols and notations. Coefficients and variables used in the alternative all-heights method equations are as follows:

\[ C_{n} = \text{Net-pressure coefficient based on } K_y [(G)(C) - (GC)] \]

\[ G = \text{Gust effect factor for rigid structures in accordance with ASCE 7 Section 26.9.1.} \]

\[ K_y = \text{Wind directionality factor in accordance with ASCE 7 Table 26-6.} \]

\[ P_{n} = \text{Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in psf (kN/m}^2). \]

❖ The primary simplification is accomplished by generating a table of net pressure coefficients (\( C_{n} \)), combining a number of parameters in a simple, yet conservative manner. These are shown in the definition of notation \( C_{n} \). Application of the tabulated net pressure coefficients reduces the number of steps required for performing a wind analysis, resulting in net wind forces that meet or exceed those calculated using the Directional Procedure of ASCE 7. A gust factor of 0.85 is used for the tabulated \( C_{n} \) values, as Section 26.9.2 of ASCE 7 permits for rigid structures.

TABLE 1609.6.2. See page 16-52.

❖ Net pressure coefficients are tabulated for both enclosed and partially enclosed structures (see Section 26.10 of ASCE 7).

For main wind-force-resisting system roofs Table 1609.6.2 refers to Conditions 1 and 2 for "windward roof slopes," which correspond to the two values of \( C_y \) listed in Figure 27.4.1-1 of ASCE 7 (Note 3 at the bottom of that figure also refers to "both conditions").

The basis is \( C_{n} = K_y [(G)(C) - (GC)] \) (see Section 1609.6.2) and the values obtained for \( C_y \). When considering the wind loads on the "windward roof slope," see Figure 27.4.1-1 of ASCE 7. On the windward side of a sloping roof for "Gable, hip roof," there
are two wind load conditions, one "upward" and the other "downward" as indicated by the two arrows. Two conditions on the windward roof slope provide two values of \( C_v \) and thereby two values of \( C_{ref} \) shown in Table 1609.6.2.

1609.6.3 Design equations. When using the alternative all-heights method, the MWFRS, and components and cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation 16-35.

\[
P_{ref} = 0.00256 V^4 K_f K_c K_y
\]

(Equation 16-35)

Design wind forces for the MWFRS shall not be less than 16 psf (0.77 kN/m²) multiplied by the area of the structure projected on a plane normal to the assumed wind direction (see ASCE 7 Section 27.4.7 for criteria). Design net wind pressure for components and cladding shall not be less than 16 psf (0.77 kN/m²) acting in either direction normal to the surface.

This section provides the formula that is used for the design wind pressure. It also incorporates the ASCE 7 minimum wind pressure for main wind-force-resisting systems and components and cladding.

1609.6.4 Design procedure. The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation 16-35.

Using Equation 16-35, the design pressures can be calculated for the main wind-force-resisting system. Similarly, the components and cladding design pressures are calculated for the various portions of the building envelope.

1609.6.4.1 Main windforce-resisting systems. The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 27.4.6.

A reference is made to the ASCE 7 figure that illustrates the design wind load cases that must be considered, specifically making mention of the torsional load cases.

1609.6.4.2 Determination of \( K_r \) and \( K_p \). Velocity pressure exposure coefficient, \( K_r \), shall be determined in accordance with ASCE 7 Section 27.3.1 and the topographic factor, \( K_p \), shall be determined in accordance with ASCE 7 Section 26.8.

1. For the windward side of a structure, \( K_p \) and \( K_r \) shall be based on height \( z \).
2. For leeward and sidewalls, and for windward and leeward roofs, \( K_p \) and \( K_r \) shall be based on mean roof height \( h \).

For the velocity pressure exposure coefficient, the user is referred to the corresponding ASCE 7 section.

For the topographic factor, the user is referred to the corresponding ASCE 7 section. Note that these are evaluated only at the mean roof height for leeward walls, sidewalls and roofs.

1609.6.4.3 Determination of net pressure coefficients, \( C_{net} \). For the design of the MWFRS and for components and cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient, \( C_{net} \).

1. The pressure coefficient, \( C_{net} \), for walls and roofs shall be determined from Table 1609.6.2.
2. Where \( C_{net} \) has more than one value, the more severe wind load condition shall be used for design.

The tabulated \( C_{net} \) values represent the sum of external and internal pressure coefficients as shown in the notation defined in Section 1609.6.2.

1609.6.4.4 Application of wind pressures. When using the alternative all-heights method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

This section clarifies how the wind pressures are applied relative to the surfaces of the building envelope.

1609.6.4.4.1 Components and cladding. Wind pressure for each component or cladding element is applied as follows using \( C_{net} \) values based on the effective wind area, \( A \), contained within the zones in areas of discontinuity of width and/or length "a," "2a" or "4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in figures in tables in ASCE 7 as referenced in Table 1609.6.2 in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include "field" (Zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas of discontinuity.
3. Where applicable, the calculated pressures at discontinuities (Zone 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

The \( C_{net} \) values for C & C pressures are separately tabulated for roof and walls. These portions of the structure are further divided into those that are in areas of discontinuities and those that are not. Effective wind area is another factor in selecting the correct C & C value. While the lateral-load-resisting system design may be controlled by earthquake forces, it is necessary to consider these C & C wind pressures, which can control the design of the component and attachments.
### TABLE 1609.6.2
NET PRESSURE COEFFICIENTS, $C_{F_{1}}$\(^a\)\(^b\)

<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>$C_{F_{1}}$ FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Enclosed</td>
<td>Partially enclosed</td>
</tr>
<tr>
<td></td>
<td>+ Internal pressure</td>
<td>- Internal pressure</td>
</tr>
<tr>
<td>Walls:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Windward wall</td>
<td>0.43</td>
<td>0.73</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.51</td>
<td>-0.21</td>
</tr>
<tr>
<td>Sidewall</td>
<td>-0.66</td>
<td>-0.35</td>
</tr>
<tr>
<td></td>
<td>Windward</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>Leeward</td>
<td>-0.85</td>
</tr>
<tr>
<td>Parapet wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Enclosed</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wind perpendicular to ridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Enclosed</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Leeward roof or flat roof</td>
<td>-0.66</td>
</tr>
<tr>
<td>Windward roof slopes:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope &lt; 2:12 (10°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-1.09</td>
<td>-0.79</td>
</tr>
<tr>
<td>Condition 2</td>
<td>-0.28</td>
<td>0.02</td>
</tr>
<tr>
<td>Slope = 4:12 (18°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-0.73</td>
<td>-0.42</td>
</tr>
<tr>
<td>Condition 2</td>
<td>-0.06</td>
<td>0.25</td>
</tr>
<tr>
<td>Slope = 5:12 (23°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-0.58</td>
<td>-0.28</td>
</tr>
<tr>
<td>Condition 2</td>
<td>0.03</td>
<td>0.34</td>
</tr>
<tr>
<td>Slope = 6:12 (27°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-0.47</td>
<td>-0.16</td>
</tr>
<tr>
<td>Condition 2</td>
<td>0.06</td>
<td>0.37</td>
</tr>
<tr>
<td>Slope = 7:12 (30°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-0.37</td>
<td>-0.06</td>
</tr>
<tr>
<td>Condition 2</td>
<td>0.07</td>
<td>0.37</td>
</tr>
<tr>
<td>Slope = 9:12 (37°)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition 1</td>
<td>-0.27</td>
<td>0.04</td>
</tr>
<tr>
<td>Condition 2</td>
<td>0.14</td>
<td>0.44</td>
</tr>
<tr>
<td>Slope = 12:12 (45°)</td>
<td>0.14</td>
<td>0.44</td>
</tr>
<tr>
<td>Wind parallel to ridge and flat roofs</td>
<td>-1.09</td>
<td>-0.79</td>
</tr>
</tbody>
</table>

Nonbuilding Structures: Chimneys, Tanks and Similar Structures:

<table>
<thead>
<tr>
<th>h/D</th>
<th>1</th>
<th>7</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Square (Wind normal to face)</td>
<td>0.99</td>
<td>1.07</td>
<td>1.53</td>
</tr>
<tr>
<td>Square (Wind on diagonal)</td>
<td>0.77</td>
<td>0.84</td>
<td>1.15</td>
</tr>
<tr>
<td>Hexagonal or Octagonal</td>
<td>0.81</td>
<td>0.97</td>
<td>1.13</td>
</tr>
<tr>
<td>Round</td>
<td>0.65</td>
<td>0.81</td>
<td>0.97</td>
</tr>
<tr>
<td>Open signs and lattice frameworks Ratio of solid to gross area</td>
<td>&lt; 0.1</td>
<td>0.1 to 0.29</td>
<td>0.3 to 0.7</td>
</tr>
<tr>
<td>Flat</td>
<td>1.45</td>
<td>1.30</td>
<td>1.16</td>
</tr>
<tr>
<td>Round</td>
<td>0.87</td>
<td>0.94</td>
<td>1.08</td>
</tr>
</tbody>
</table>

(continued)
<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>C_{net} FACTOR</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Enclosed</td>
<td>Partially enclosed</td>
</tr>
<tr>
<td>Roof elements and slopes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gable of hipped configurations (Zone 1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.58</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.00</td>
<td>-1.32</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.92</td>
<td>-1.23</td>
</tr>
<tr>
<td>Overhang: Flat &lt; Slope &lt; 6:12 (27°) See ASCE 7 Figure 30.4-2A Zone 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.36</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.94</td>
<td></td>
</tr>
<tr>
<td>6:12 (27°) &lt; Slope &lt; 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.92</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.83</td>
<td>1.15</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.00</td>
<td>-1.32</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.83</td>
<td>-1.15</td>
</tr>
<tr>
<td>Monosloped configurations (Zone 1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 7:12 (30°) See ASCE 7 Figure 30.4-5B Zone 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.49</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.26</td>
<td>-1.57</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.09</td>
<td>-1.40</td>
</tr>
<tr>
<td>Tall flat-topped roofs h &gt; 60 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 2:12 (10°) (Zone 1) See ASCE 7 Figure 30.8-1 Zone 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.34</td>
<td>-1.66</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.92</td>
<td>-1.23</td>
</tr>
<tr>
<td>Gable or hipped configurations at ridges, eaves and rakes (Zone 2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.58</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.68</td>
<td>-2.00</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.17</td>
<td>-1.49</td>
</tr>
<tr>
<td>Overhang for Slope Flat &lt; Slope &lt; 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.87</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.87</td>
<td></td>
</tr>
<tr>
<td>6:12 (27°) &lt; Slope &lt; 12:12 (45°) Figure 30.4-2C</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.92</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.83</td>
<td>1.15</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.17</td>
<td>-1.49</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.00</td>
<td>-1.32</td>
</tr>
<tr>
<td>Overhang for 6:12 (27°) &lt; Slope &lt; 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.53</td>
<td></td>
</tr>
</tbody>
</table>

(continued)
### Table 1609.6.2—continued

#### NET PRESSURE COEFFICIENTS, $C_{net}$

<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>$C_{net}$ FACTOR</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Roof elements and slopes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Monosloped configurations at ridges, eaves and rakes (Zone 2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 7:12 (30°) See ASCE 7 Figure 30.4-5B Zone 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.49</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.51</td>
<td>-1.83</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.43</td>
<td>-1.74</td>
</tr>
<tr>
<td>Tall flat topped roofs $h &gt; 60$ feet</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 2:12 (10°) (Zone 2) See ASCE 7 Figure 30.8-1 Zone 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-2.11</td>
<td>-2.42</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.51</td>
<td>-1.83</td>
</tr>
<tr>
<td>Gable or hipped configurations at corners (Zone 3) See ASCE 7 Figure 30.4-2B Zone 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 6:12 (27°)</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.58</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-2.53</td>
<td>-2.85</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.85</td>
<td>-2.17</td>
</tr>
<tr>
<td>Overhang for Slope Flat &lt; Slope &lt; 6:12 (27°) See ASCE 7 Figure 30.4-2B Zone 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-3.15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-2.13</td>
<td></td>
</tr>
<tr>
<td>6:12 (27°) &lt; 12:12 (45°) See ASCE 7 Figure 30.4-2C Zone 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.92</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.83</td>
<td>1.15</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.17</td>
<td>-1.49</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.00</td>
<td>-1.32</td>
</tr>
<tr>
<td>Overhang for 6:12 (27°) &lt; Slope &lt; 12:12 (45°)</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.53</td>
<td></td>
</tr>
<tr>
<td>Monosloped Configurations at corners (Zone 3) See ASCE 7 Figure 30.4-5B Zone 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 7:12 (30°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.49</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-2.62</td>
<td>-2.93</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.85</td>
<td>-2.17</td>
</tr>
<tr>
<td>Tall flat topped roofs $h &gt; 60$ feet</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Flat &lt; Slope &lt; 2:12 (10°) (Zone 3) See ASCE 7 Figure 30.8-1 Zone 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-2.87</td>
<td>-3.19</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-2.11</td>
<td>-2.42</td>
</tr>
<tr>
<td>Wall Elements: $h = 60$ feet (Zone 4) See ASCE 7 Figure 30.4-1</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>1.00</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>0.75</td>
<td>1.06</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.09</td>
<td>-1.40</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.83</td>
<td>-1.15</td>
</tr>
<tr>
<td>Wall Elements: $h &gt; 60$ feet (Zone 4) See ASCE 7 Figure 30.8-1 Zone 4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>20 square feet or less</td>
<td>0.92</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>0.66</td>
<td>0.98</td>
</tr>
</tbody>
</table>

(continued)
### TABLE 1099.6.2—continued
**NET PRESSURE COEFFICIENTS, C<sub>net</sub><sup>a,b</sup>**

<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>C&lt;sub&gt;net&lt;/sub&gt; FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. Components and cladding not in areas of discontinuity—walls and parapets</td>
<td>Negative</td>
<td>-0.92</td>
</tr>
<tr>
<td></td>
<td>20 square feet or less</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.75</td>
</tr>
<tr>
<td></td>
<td>Parapet Walls</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Positive</td>
<td>2.87</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>-1.68</td>
</tr>
<tr>
<td></td>
<td>Wall elements: h ≤ 60 feet (Zone 5) Figure 30.4-1</td>
<td>Enclosed</td>
</tr>
<tr>
<td></td>
<td>Positive</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10 square feet or less</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>-1.34</td>
</tr>
<tr>
<td></td>
<td>10 square feet or less</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.83</td>
</tr>
<tr>
<td></td>
<td>Wall elements: h &gt; 60 feet (Zone 5) See ASCE 7 Figure 30.8-1 Zone 4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Positive</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>20 square feet or less</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>-1.68</td>
</tr>
<tr>
<td></td>
<td>20 square feet or less</td>
<td></td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.00</td>
</tr>
<tr>
<td></td>
<td>Parapet walls</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Positive</td>
<td>3.64</td>
</tr>
<tr>
<td></td>
<td>Negative</td>
<td>-2.45</td>
</tr>
</tbody>
</table>

For SI: 1 foot = 0.3048 m, 1 square foot = 0.0929 m², 1 degree = 0.0175 rad.
a. Linear interpolation between values in the table is permitted.
b. Some C<sub>net</sub> values have been grouped together. Less conservative results may be obtained by applying ASCE 7 provisions.

### SECTION 1610
**SOIL LATERAL LOADS**

1610.1 **General.** Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

**Exception:** Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

- Nominal loads attributable to lateral earth pressures are determined in accordance with this section. This section provides lateral loads for various soil types. This section requires that foundation and retaining walls be designed to be capable of resisting the lateral soil loads specified by Table 1610.1 where a specific soil investigation has not been performed.

Consideration must be given to additional lateral soil pressures due to surcharge loads that result from sloping backfill, driveways or parking spaces that are close to a foundation wall, as well as the foundation of an adjacent structure.

**TABLE 1610.1.** See page 16-56.

- The table lists at-rest and active soil pressures for a number of different types of moist soils. The basis of the soil classification into the various types listed is ASTM D 2487. Soils identified by Note b in Table 1610.1 have unpredictable characteristics. These are called expansive soils. Because of their ability to absorb water, they shrink and swell to a higher degree than other soils. As expansive soils swell, they are capable of exerting large forces on soil-retaining structures; thus, these types of soils are not to be used as backfill.

### SECTION 1611
**RAIN LOADS**

1611.1 **Design rain loads.** Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow. The design rainfall shall be based on the 100-year
hourly rainfall rate indicated in Figure 1611.1 or on other rainfall rates determined from approved local weather data.

\[ R = 5.2(d_i + d_s) \]  \hspace{1cm} \text{(Equation 16-36)}

For SI: \[ R = 0.0098(d_i + d_s) \]

where:

- \( d_i \) = Additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow (i.e., the hydraulic head), in inches (mm).
- \( d_s \) = Depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (i.e., the static head), in inches (mm).
- \( R \) = Rain load on the undeflected roof, in psf (kN/m²).

When the phrase “undeflected roof” is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

The nominal rain load, \( R \), is determined in accordance with this section. It represents the weight of accumulated rainwater, assuming a blockage of the primary roof drainage system. The design of the roof drainage systems must be in accordance with Chapter 11 of the International Plumbing Code® (IPC®). The primary roof drainage system can include roof drains, leaders, conductors and horizontal storm drains within the structure. Drainage system design is based on a specified design rainfall intensity, as well as the roof area it drains. The criteria for sizing the components of the drainage system are provided in Section 1106 of the IPC. Where the building is configured such that water will not collect on the roof there is no requirement for a secondary drainage system [see Figure 1611.1(1)]. Likewise, there would be no rain load required in the design of the roof.

It is not uncommon to find that roof drains have become blocked by debris, leading to ponding of rainwater where the roof construction is conducive to retaining water. While the objective of providing roof drainage is typically to prevent the accumulation of water, the code also recognizes controlled drainage systems that are engineered to retain rainwater (see Section 1101.3). The important point is that wherever the potential exists for the accumulation of rainwater on a roof, whether it is intentional or otherwise, the roof must be designed for this load. Furthermore, Section 1101.7 of the IPC requires the maximum depth of water to be determined, assuming all primary roof drainage to be blocked. The water will rise above the primary roof drain until it reaches the elevation of the roof edge, scuppers or another serviceable drain. At the design rainfall intensity, this depth will be based on the flow rate of the secondary drainage system. This depth, referred to as the hydraulic head, can be determined from Table 1611.1(2) for various types of drains and flow rates. Its use is illustrated in the example on page 16-65. Section 1108 of

<table>
<thead>
<tr>
<th>DESCRIPTION OF BACKFILL MATERIAL*</th>
<th>UNIFIED SOIL CLASSIFICATION</th>
<th>DESIGN LATERAL SOIL LOAD* (pound per square foot per foot of depth)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well-graded, clean gravels; gravel-sand mixes</td>
<td>GW</td>
<td>30</td>
</tr>
<tr>
<td>Poorly graded clean gravels; gravel-sand mixes</td>
<td>GP</td>
<td>30</td>
</tr>
<tr>
<td>Silty gravels, poorly graded gravel-sand mixes</td>
<td>GM</td>
<td>40</td>
</tr>
<tr>
<td>Clayey gravels, poorly graded gravel-and-clay mixes</td>
<td>GC</td>
<td>45</td>
</tr>
<tr>
<td>Well-graded, clean sands; gravelly sand mixes</td>
<td>SW</td>
<td>30</td>
</tr>
<tr>
<td>Poorly graded clean sands; sand-gravel mixes</td>
<td>SP</td>
<td>30</td>
</tr>
<tr>
<td>Silty sands, poorly graded sand-silt mixes</td>
<td>SM</td>
<td>45</td>
</tr>
<tr>
<td>Sand-silt clay mix with plastic fines</td>
<td>SM-SC</td>
<td>45</td>
</tr>
<tr>
<td>Clayey sands, poorly graded sand-clay mixes</td>
<td>SC</td>
<td>60</td>
</tr>
<tr>
<td>Inorganic silts and clayey silts</td>
<td>ML</td>
<td>45</td>
</tr>
<tr>
<td>Mixture of inorganic silt and clay</td>
<td>ML-CL</td>
<td>60</td>
</tr>
<tr>
<td>Inorganic clays of low to medium plasticity</td>
<td>CL</td>
<td>60</td>
</tr>
<tr>
<td>Organic silts and silt clays, low plasticity</td>
<td>OL</td>
<td>Note b</td>
</tr>
<tr>
<td>Inorganic clayey silts, elastic silts</td>
<td>MH</td>
<td>Note b</td>
</tr>
<tr>
<td>Inorganic clays of high plasticity</td>
<td>CH</td>
<td>Note b</td>
</tr>
<tr>
<td>Organic clays and silty clays</td>
<td>OH</td>
<td>Note b</td>
</tr>
</tbody>
</table>

For SI: 1 pound per square foot per foot of depth = 0.157 kPa/m, 1 foot = 304.8 mm.

a. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.

b. Unsuitable as backfill material.

c. The definition and classification of soil materials shall be in accordance with ASTM D 2487.
the IPC specifically requires a secondary roof drainage system where the building construction extends above the roof at the perimeter. This applies to parapet walls, stepped buildings or any other construction that would allow rainwater to pond on the roof. The sizing of a secondary drainage system is identical to the process used for the primary system. Instead of using a "piped" secondary system, designers may prefer to install scuppers to allow rainwater to overflow the roof. Examples of both types of secondary systems are shown in Figure 1611.1(3). Also note that the IPC requires a secondary system to be completely separate and to discharge above grade. Since the secondary system serves as an emergency backup, requiring it to discharge above grade provides a means of signaling that there is a blockage of the primary drainage system.

Some roof failures have been attributed to the increased loads from ponding water. This section requires the roof to be capable of resisting the maximum water depth that can occur if the primary means of roof drainage becomes blocked. Blockages are typically caused by debris at the inlet to the primary roof drains, but they can occur anywhere along the primary piping system, such as an under-slab pipe collapse. Computation of rain load, \( R \), is in accordance with Equation 16-36. The coefficient of that equation is merely the conversion of the unit weight of water to an equivalent unit load per inch of water depth as Figure 1611.1(4) illustrates. Two variables are considered to determine rain load: the depth of the water on the undeflected roof, as measured from the low point elevation to the inlet elevation of the secondary drain; and the additional depth of water at the secondary drainage flow, respectively referred to as static head and hydraulic head. The sum of these depths is the design depth for computing rain load, \( R \), as indicated in Equation 16-36. An example of the

![Diagram](image)

**Figure 1611.1(1)**
SECONDARY ROOF DRAINAGE NOT REQUIRED


<table>
<thead>
<tr>
<th>DRAINAGE SYSTEM</th>
<th>FLOW RATE (gpm)</th>
<th>Depth of water above drain inlet (hydraulic head) (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>4-inch-diameter drain</td>
<td>80</td>
<td>170</td>
</tr>
<tr>
<td>6-inch-diameter drain</td>
<td>100</td>
<td>190</td>
</tr>
<tr>
<td>8-inch-diameter drain</td>
<td>125</td>
<td>230</td>
</tr>
<tr>
<td>6-inch-wide, open-top scupper</td>
<td>18</td>
<td>50</td>
</tr>
<tr>
<td>24-inch-wide, open-top scupper</td>
<td>72</td>
<td>200</td>
</tr>
<tr>
<td>6-inch-wide, 4-inch-high, closed-top scupper</td>
<td>18</td>
<td>50</td>
</tr>
<tr>
<td>24-inch-wide, 4-inch-high, closed-top scupper</td>
<td>72</td>
<td>200</td>
</tr>
<tr>
<td>6-inch-wide, 6-inch-high, closed-top scupper</td>
<td>18</td>
<td>50</td>
</tr>
<tr>
<td>24-inch-wide, 6-inch-high, closed-top scupper</td>
<td>72</td>
<td>200</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 gallon per minute = 3.785 L/min.
Source: Factory Mutual Engineering Corp. Loss Prevention Data 1-54.

**Table 1611.1(2)**
FLOW RATE, IN GALLONS PER MINUTE, OF VARIOUS ROOF DRAINS AT VARIOUS WATER DEPTHS AT DRAIN INLETS (INCHES)

2012 INTERNATIONAL BUILDING CODE® COMMENTARY  16-57
computation of rain load is provided in the following example [also see Figure 1611.1(5)].

EXAMPLE
Rain Load on Roof with Overflow Scuppers

Given:
Primary roof drain and scupper shown in Figure 1611.1(5).
Static head, \( d_s \) = 7 inches (178 mm)
Tributary area, \( A \) = 5,400 square feet (502 m²)
Rainfall rate, \( i \) = 2.5 inches/hour
= 0.208333 feet/hour
= 0.0635 m/hr

Determine:
Hydraulic head, \( d_h \)
Rain load, \( R \)

Calculate required flow rate, \( Q \), at scupper in gallons per minute (gpm).
\[ Q = A \times i = 5,400 \text{ square feet} \times 0.2083 \text{ feet/hour} \]
= 1125 cubic feet/hour (31.9 m³/hr)
= 18.75 cubic feet/minute (0.531 m³/min)
= 140.25 gpm (531 L/min)

Look up hydraulic head using Table 1611.1(2).

For 6 inches wide 6 inches (152 mm) high scupper, find 140 gpm. (530 L/m)
\( d_s = 4 \) inches (102 mm)

Determine rain load, \( R \)
Total head = \( d_s + d_h \) = 11 inches (279 mm)
\[ R = 5.2 (d_s + d_h) \]
\[ R = 57.2 \text{ psf (2.74 kN/m²)} \]

FIGURE 1611.1. See page 16-60.

Including these IPC maps in the code provides the structural designer with design criteria that are essential for determining the rain load. Figure 1611.1 consists of five maps for various regions of the country. This figure provides the rainfall rates for a storm of 1-hour duration that has a 100-year return period. Rainfall rates indicate the maximum rate of rainfall within the given period of time occurring at the stated frequency. For example, the map indicates a rainfall rate of 2.1 inches (53 mm) per hour for Burlington, Vermont. Thus, it is predicted that it will rain 2.1 inches (53 mm) within 1 hour once every 100 years. The rainfall rates are calculated by a statistical analysis of weather records. Because the statistics are based on previous or historical weather conditions, it is conceivable to have two 100-year storms in one week’s time. The probability of this occurring, however, is very low.
Figure 1611.1(4)
EQUATION 16-35 COEFFICIENT

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm,
1 cubic foot = 0.02832 m³, 1 pound = 0.454 kg,
1 pound per square foot = 47.88 Pa.

Figure 1611.1(5)
RAIN LOAD EXAMPLE

For SI: 1 inch = 25.4 mm.
[P] FIGURE 1611.1
100-YEAR, 1-HOUR RAINFALL (INCHES) WESTERN UNITED STATES

For SI: 1 inch = 25.4 mm.
[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) CENTRAL UNITED STATES

For SI: 1 inch = 25.4 mm.
For SI: 1 inch = 25.4 mm.
[P] FIGURE 1611.1—continued
100-YEAR, 1-HOUR RAINFALL (INCHES) ALASKA

For SI: 1 inch = 25.4 mm.
For SI: 1 inch = 25.4 mm.
1611.2 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 8.4 of ASCE 7.

- In roofs lacking sufficient framing stiffness, a condition known as "ponding instability" can occur where increasingly larger deflections caused by the continued accumulation of rainwater are large enough to overload the structure and result in a roof collapse. This must be countered by providing adequate stiffness in order to prevent increasingly larger deflections due to the buildup of rainwater. Another means to minimize the accumulation of rainwater is to camber the roof framing. This section requires a check for ponding instability at susceptible bays (see definition in Section 202). A ponding instability check is to be made assuming the primary roof drains are blocked. The determination of ponding instability is typically done by an iterative structural analysis where the incremental deflection is determined and the resulting increase in rainfall load from the deflection is added to the original rainfall load.

1611.3 Controlled drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from Section 1611.1. Such roofs shall also be checked for ponding instability in accordance with Section 1611.2.

- Controlled drainage is the limitation of the drainage flow rate to a rate that is less than the rainfall rate such that the depth of the water intentionally builds up on the roof during a design rainfall. Controlled flow roof drain systems must be designed in accordance with Section 1111 of the IRC. A secondary roof drain system is needed to limit the buildup of water to a specific depth for roof design. The depth of water on the roof is also to include the depth of the water above the inlet of the secondary drain when the design flow rate is reached. Consideration of the effect of the accumulated rainwater is identical to assuming a blockage in the primary drainage system as discussed under Section 1611.1.

SECTION 1612 FLOOD LOADS

1612.1 General. Within flood hazard areas as established in Section 1612.3, all new construction of buildings, structures and portions of buildings and structures, including substantial improvement and restoration of substantial damage to buildings and structures, shall be designed and constructed to resist the effects of flood hazards and flood loads. For buildings that are located in more than one flood hazard area, the provisions associated with the most restrictive flood hazard area shall apply.

- This section addresses requirements for all buildings and structures in flood hazard areas. These areas are commonly referred to as "flood plains" and are shown on a community's Flood Insurance Rate Map (FIRM) prepared by FEMA or on another adopted flood hazard map. Code users should be aware that floods often affect areas outside the flood hazard area boundaries shown on FIRMs and can exceed the base flood elevation.

Through the adoption of the code, communities meet a significant portion of the flood plain management regulation requirements necessary to participate in the National Flood Insurance Program (NFIP). To participate in the NFIP, a jurisdiction must adopt regulations that at least meet the requirements of federal regulations in Section 60.3 of 44 CFR. This requirement can be satisfied by adopting the International Building Code® (IBC®) (including Appendix G) and the International Residential Code® (IRC®). [The International Existing Building Code® (IEBC®) also has provisions that are consistent with the NFIP.] If Appendix G (Flood-resistant Construction) is not enforced, its provisions must be captured in a companion flood plain management ordinance adopted by the community. The NFIP requires communities to regulate all development in flood hazard areas. Section 201.2 of Appendix G defines "Development" as "any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures; temporary or permanent storage of materials; mining; dredging; filling; grading; paving; excavations; operations and other land-disturbing activities."

The NFIP was established to reduce flood losses, to better indemnify individuals from flood losses and to reduce federal expenditures for disaster assistance. A community that has been determined to have flood hazard areas elects to participate in the NFIP to protect health, safety and property, so that its citizens can purchase federally backed flood insurance. FEMA administers the NFIP, which includes monitoring community compliance with the flood plain management requirements of the NFIP.

New buildings and structures, and substantial improvements to existing buildings and structures, are to be designed and constructed to resist flood forces to minimize damage. Flood forces include flotation, lateral (hydrostatic) pressures, moving water (hydrodynamic) pressures, wave impact and debris impact. Flood-related hazards may include erosion and scour.

Many states and communities have elected to regulate flood plain development to a higher standard than the minimum required to participate in the NFIP. Communities considering using the code and other International Codes® to meet the flood plain manage-
ment requirements of the NFIP are advised to consult with their state NFIP coordinator or the appropriate FEMA regional office.

If located in flood hazard areas, buildings and structures that are damaged by any cause are to be examined by the building official to determine if the damage constitutes substantial damage, in which the cost to repair or restore the building or structure to its predamaged condition equals or exceeds 50 percent of its market value before the damage occurred. All substantial improvements and repairs of buildings and structures that are substantially damaged must meet the flood-resistant provisions of the code. For additional guidance, see FEMA P-758, Substantial Improvement/Substantial Damage Desk Reference.

Some buildings are proposed to be located such that either they are in more than one flood zone or only a portion of the building is in a flood zone. Where this occurs, the entire building or structure is required to be designed and constructed according to the requirements of the more restrictive flood zone. For example, if a building is partially in a flood hazard area subject to high-velocity wave action (V zone), then the entire building must meet the requirements for that area. Similarly, if a building is partially in a flood hazard area and partially out of the mapped flood plain, then the entire building must be flood resistant.

For additional guidance on how to use the International Code® to participate in the NFIP, see Reducing Flood Losses through the International Codes: Meeting the Requirements of the National Flood Insurance Program.

1612.2 Definitions. The following terms are defined in Chapter 2:

BASE FLOOD.
BASE FLOOD ELEVATION.
BASEMENT.
DESIGN FLOOD.
DESIGN FLOOD ELEVATION.
DRY FLOODPROOFING.
EXISTING CONSTRUCTION.
EXISTING STRUCTURE.
FLOOD or FLOODING.
FLOOD DAMAGE-RESISTANT MATERIALS.
FLOOD HAZARD AREA.
FLOOD HAZARD AREA SUBJECT TO HIGH-VELOCITY WAVE ACTION.
FLOOD INSURANCE RATE MAP (FIRM).
FLOOD INSURANCE STUDY.
FLOODWAY.
LOWEST FLOOR.
SPECIAL FLOOD HAZARD AREA.

START OF CONSTRUCTION.

SUBSTANTIAL DAMAGE.

SUBSTANTIAL IMPROVEMENT.

Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with flood requirements. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

1612.3 Establishment of flood hazard areas. To establish flood hazard areas, the applicable governing authority shall adopt a flood hazard map and supporting data. The flood hazard map shall include, at a minimum, areas of special flood hazard as identified by the Federal Emergency Management Agency in an engineering report entitled "The Flood Insurance Study for [INSERT NAME OF JURISDICTION]," dated [INSERT DATE OF ISSUANCE], as amended or revised with the accompanying Flood Insurance Rate Map (FIRM) and Flood Boundary and Floodway Map (FBFM) and related supporting data along with any revisions thereto. The adopted flood hazard map and supporting data are hereby adopted by reference and declared to be part of this section.

Flood maps and studies are prepared by FEMA and are to be used as a community’s official map, unless the community chooses to adopt a map that shows more extensive flood hazard areas. Most communities have multiple flood map panels, all of which should be listed in the adopting ordinance by panel number and date so that the appropriate effective map is used.

From time to time, FEMA’s flood plain maps and studies may be revised and republished. In recent years, revised FIRMs have been produced in a digital format (referred to as Digital FIRMs or DFIRMs). Communities that prefer to cite the digital data should obtain a legal opinion. DFIRMs are registered to the primary coordinate system of the state or community. FEMA advises that the horizontal location of flood hazard areas relative to specific sites should be determined using the coordinate grid rather than planimetric base map features such as streets.

When maps are revised and flood hazard areas are changed, FEMA involves the community and provides a formal opportunity to review the documents. Once the revisions are finalized, FEMA requires the community to adopt the new maps. Communities may be advised to minimize having to adopt each revision by referencing the date of the original map and study and all future revisions. This is a method by which subsequent revisions to flood maps and studies may be adopted administratively without requiring legislative action on the part of the community. Communities will need to determine whether this “adoption by reference” approach is allowed under their state’s enabling authority and due process requirements. If not allowed, communities are to follow their state’s requirements, which typically require public
1612.3.1 Design flood elevations. Where design flood elevations are not included in the flood hazard areas established in Section 1612.3, or where floodways are not designated, the building official is authorized to require the applicant to:

1. Obtain and reasonably utilize any design flood elevation and floodway data available from a federal, state or other source; or
2. Determine the design flood elevation and/or floodway in accordance with accepted hydrologic and hydraulic engineering practices used to define special flood hazard areas. Determinations shall be undertaken by a registered design professional who shall document that the technical methods used reflect currently accepted engineering practice.

The purpose of this provision is to clarify how design flood elevations are to be determined for those flood areas shown on community flood hazard maps that do not have the flood elevation already specified. Section 107.2 requires that the construction documents submitted are to be accompanied by a site plan, which includes flood hazard areas, floodways or design flood elevations, as applicable. While flood elevations are often available, a large percentage of areas that are mapped as special flood hazard areas by the NFIP do not have flood elevations or do not have floodway designations. This section clarifies the authority of the building official to require use of data, which may be obtained from other sources, or to require the applicant to develop flood hazard data, and is based on the NFIP regulation in 44 CFR §60.3(b)(4).

1612.3.2 Determination of impacts. In riverine flood hazard areas where design flood elevations are specified but floodways have not been designated, the applicant shall provide a floodway analysis that demonstrates that the proposed work will not increase the design flood elevation more than 1 foot (305 mm) at any point within the jurisdiction of the applicable governing authority.

This section requires a floodway analysis to determine impacts. Development in riverine flood plains can increase flood levels and loads on other properties, especially if it occurs in areas known as "floodways" that are reserved to convey flood flows. Commercial software for these analyses is readily available and FEMA provides software and technical guidance at http://www.fema.gov/plan/prevent/fhm frm_soft.shtml.

This section provides consistency with the NFIP, which requires applicants to demonstrate whether their proposed work will increase flood levels in floodplains where the NFIP's FIRM shows base flood elevations, but floodways are not shown. A small percentage of flood plains where FEMA has specified base flood elevations do not have designated floodways.

1612.4 Design and construction. The design and construction of buildings and structures located in flood hazard areas, including flood hazard areas subject to high-velocity wave action, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

- FEMA uses multiple designations for flood hazard areas shown on each FIRM, including A, AO, AH, A1-30, AE, A99, AR, AR/A1-30, AR/AE, AR/AO, AR/AH, AR/A, VO or V1-30, VE and V. Along many open coasts and lake shorelines where wind-driven waves are predicted, the flood hazard area is commonly referred to as the "V zone." Flood hazard areas that are inland of areas subject to high-velocity wave action and flood hazard areas along rivers and streams are commonly referred to as "A zones." Due to waves, the flood loads in areas subject to high-velocity wave action differ from those in other flood hazard areas. Some recent FIRMS in coastal communities show the "Limit of Moderate Wave Action," which is the inland extent of the 1.5-foot (457 mm) wave height. In ASCE 7 and ASCE 24, as well as in common usage, the area between the V zone and Limit of Moderate Wave Action is called "Coastal A Zone."

ASCE 24 outlines in detail the specific requirements that are to be applied to buildings and structures in all flood hazard areas. Communities that have AR zones and A99 zones shown on their FIRMs should consult with the appropriate state agency or FEMA regional office for guidance on the requirements that apply. AR zones are areas that result from the decertification of a previously accredited flood protection system (such as a levee) that is determined to be in the process of being restored to provide base flood protection. A99 zones are areas subject to inundation by the 1-percent-annual-chance flood event, but which will ultimately be protected upon completion of an under-construction federal flood protection system.

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a registered design professional and submitted to the building official:

1. For construction in flood hazard areas not subject to high-velocity wave action:
   1.1. The elevation of the lowest floor, including the basement, as required by the lowest floor elevation inspection in Section 110.3.3.
   1.2. For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 26.2.1 of ASCE 24, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 26.6.2.2 of ASCE 24.
1.3. For dry floodproofed nonresidential buildings, *construction documents* shall include a statement that the dry floodproofing is designed in accordance with ASCE 24.

2. For construction in flood hazard areas subject to high-velocity wave action:

2.1. The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation inspection in Section 110.3.3.

2.2. *Construction documents* shall include a statement that the building is designed in accordance with ASCE 24, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Chapter 16.

2.3. For breakaway walls designed to have a resistance of more than 20 psf (0.96 kN/m²) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

The NFIP requires that certain documentation be submitted in order to demonstrate compliance with provisions of the code that cannot be easily verified during a site inspection. The most common, described in Item 1.1, is the documentation of the lowest floor elevation. It provides documents that the lowest floor is at or above the required minimum elevation. Elevation is one of the most important aspects of flood-resistant construction and is a significant factor used to determine flood insurance premium rates. FEMA Form 81-31, *Elevation Certificate*, which includes illustrations and instructions, is recommended (download from www.fema.gov/business/nfip/elvinst.shtml). Building owners need elevation certificates to obtain NFIP flood insurance, and insurance agents use the certificates to compute the proper flood insurance premium rates.

The criteria for the minimum number and size of flood openings to allow free inflow and outflow of floodwaters under all types of flood conditions are set in ASCE 24. The statement described in Item 1.2 is required in the construction documents if the engineerings opened are used (see Section 2.6.2.2 of ASCE 24); this statement is not required if nonengineered (prescriptive) openings meet the criteria of Section 2.6.2.1 of ASCE 24. For further guidance, refer to FEMA TB #1, *Openings in Foundation Walls and Walls of Enclosures Below Elevated Buildings in Special Flood Hazard Areas*.

The statement described in Item 1.3 is to be included in the construction documents for nonresidential buildings that are designed to be dry floodproofed. It is important to note that dry floodproofing is allowed only for nonresidential buildings and structures that are located in flood hazard areas not subject to high-velocity wave action. The registered design professional who seals the construction documents is indicating that, based upon development or review of the structural design, specifications and plans for construction, the design and methods of construction are in accordance with accepted standards of practice in ASCE 24 to meet the following provisions: (1) the structure, together with attendant utilities and sanitary facilities, is water tight to the floodproofed design elevation indicated with walls that are substantially impermeable to the passage of water and (2) all structural components are capable of resisting hydrostatic and hydrodynamic flood forces, including the effects of buoyancy and anticipated debris impact forces. The use of FEMA Form 81-65, *Floodproofing Certificate*, is recommended (download from http://www.fema.gov/plan/prevent/floodplain/nfipkeywordsfloodproofing_certificate.shtml). This certificate is used by insurance agents to determine NFIP flood insurance premium rates for dry floodproofed nonresidential buildings. For further guidance, refer to FEMA TB #3, *Nonresidential Floodproofing—Requirements and Certification for Buildings Located in Special Flood Hazard Areas*.

Certain documentation must be submitted in order to demonstrate compliance with provisions of the code that cannot be verified readily during a site inspection. The most common, in Item 2.1, provides evidence that the bottom of the lowest horizontal members of buildings constructed in flood hazard areas subject to high-velocity wave action (V zones) are elevated to or above the minimum required height. Buildings located in flood hazard areas subject to high-velocity wave action and winds are expected to experience significant flood and wind loads simultaneously. FEMA and coastal communities report significant damage to buildings that are not built to current code. The statement described in Item 2.2 is included in the construction documents to indicate that the design meets the flood load provisions of ASCE 24 and other load requirements by this chapter.

The documentation described in Item 2.3 is used only for specific situations in which properly elevated buildings in flood hazard areas subject to high-velocity wave action have enclosures beneath them, and then only if the walls of the enclosures are designed to resist more than 20 psf (0.96 kN/m²) determined using ASD. Because breakaway walls will fail under flood conditions, building materials can become water-borne debris that may damage adjacent buildings. Refer to FEMA TB #5, *Free-of-Obstruction Requirements for Buildings Located in Coastal High-hazard Areas*, and FEMA TB #8, *Design and Construction Guidance for Breakaway Walls Below Elevated Buildings in Coastal High Hazard Areas*. 

16-68

2012 INTERNATIONAL BUILDING CODE® COMMENTARY
SECTION 1613
EARTHQUAKE LOADS

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:
1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S_p$, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

These code provisions provide the requirements essential to determining a building’s seismic design category. The balance of the earthquake load provisions are contained in the ASCE 7 load standard. The chapters that are noted as excluded from the ASCE 7 referenced standard are those that can create conflicts with Chapters 17 through 23. The balance of ASCE 7 earthquake load provisions are as follows:

Chapter Subject
11 Seismic Design Criteria
12 Seismic Design Requirements for Building-Structures
13 Seismic Design Requirements for Nonstructural Components
15 Seismic Design Requirements for Non-building Structures
16 Seismic Response History Procedures
17 Seismic Design Requirements for Seismically Isolated Structures
18 Seismic Design Requirements for Structures with Damping Systems
19 Soil Structure Interaction for Seismic Design
20 Site Classification Procedure for Seismic Design
21 Site-specific Ground Motion Procedures for Seismic Design
22 Seismic Ground Motion and Long Period Transition Maps
23 Seismic Design Reference Documents

These seismic requirements are based, for the most part, on the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures. A new set of NEHRP recommended provisions has been prepared by the BSSC approximately every three years since the first edition in 1985. The code uses the NEHRP recommended provisions as the technical basis for seismic design requirements because of the nationwide input into the development of these design criteria. The NEHRP recommended provisions present up-to-date criteria for the design and construction of buildings subject to earthquake ground motions that are applicable anywhere in the nation. The requirements are intended to minimize the hazard to life for all buildings, increase the expected performance of higher occupancy buildings as compared to ordinary buildings and improve the capability of essential facilities to function during and after an earthquake. These minimum criteria are considered to be prudent and economically justified for the protection of life safety in buildings subject to earthquakes. Achieving the intended performance, however, depends on a number of factors, including the type of structural framing type, configuration, construction materials and as-built details of construction.


There are four exceptions to seismic design requirements included in this section. The following discussion addresses each of the exceptions.

Exception 1 exempts detached one- and two-family dwellings under two conditions. The first applies to structures assigned to Seismic Design Category A, B or C (meaning that both $S_{ps} < 0.5g$ and $S_{oi} < 0.2g$). The second is for structures having a value for mapped short-period spectral response acceleration, $S_p$, less than 0.4g. The latter condition is derived from the ASCE 7 seismic provisions. Since it is based solely on the mapped value for short periods, it may allow structures to qualify more directly for this exception than will the first condition. In other words, the value of $S_p$ should be checked first. If the structure qualifies based on $S_p$ then it is not necessary to check the seismic design category.

Exception 2 exempts conventional light-frame wood construction from the seismic requirements in this chapter. It should be noted that the limitations for conventional light-frame wood construction are included in Section 2308.2. There is no limitation on the use of the structure, except that Risk Category IV
structures are not permitted to be constructed using the conventional light-frame wood construction provisions. Similarly, irregular portions of Seismic Design Category D and E structures are not permitted to be constructed using conventional light-frame wood construction (see commentary, Section 2308.12). The conventional light-frame wood construction provisions are deemed to provide equivalent seismic resistance as compared to construction designed in accordance with the requirements of this chapter based on the history of such conventional construction.

In Exception 3, agricultural buildings are exempt because they present a minimal life-safety hazard due to the low probability of human occupancy.

Exception 4 corresponds to an ASCE 7 exception regarding structures that are covered under other regulations, making it unnecessary to comply with the earthquake load requirements of the code.

1613.2 Definitions. The following terms are defined in Chapter 2:

- Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with earthquake loads. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

DESIGN EARTHQUAKE GROUND MOTION.
MECHANICAL SYSTEMS.
ORTHOGONAL.
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATION.
SEISMIC DESIGN CATEGORY.
SEISMIC FORCE-RESISTING SYSTEM.
SITE CLASS.
SITE COEFFICIENTS.

1613.3 Seismic ground motion values. Seismic ground motion values shall be determined in accordance with this section.

- The design earthquake ground motion levels determined in this section may result in damage, both structural and nonstructural, from the high stresses that occur because of the dynamic nature of seismic events. For most structures, damage from the design earthquake ground motion would be repairable, but might be so costly as to make it economically undesirable. For essential facilities, it is expected that damage from the design earthquake ground motion would not be so severe as to prevent continued occupancy and function of the facility. For ground motions greater than the design levels, the intent is that there be a low likelihood of structural collapse.

1613.3.1 Mapped acceleration parameters. The parameters $S$ and $S_z$ shall be determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(6). Where $S$ is less than or equal to 0.04 and $S_z$ is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A. The parameters $S$ and $S_z$ shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for American Samoa.

- The mapped maximum considered earthquake spectral response accelerations at 0.2-second period ($S$) and 1-second period ($S_z$) for a particular site are to be determined from Figures 1613.3.1(1) through (6). Where a site is between contours, as would usually be the case, straight-line interpolation or the value of the higher contour may be used. Areas that are considered to have a low seismic risk based solely on the mapped ground motions are placed directly into Seismic Design Category A.

The mapped maximum considered earthquake spectral response accelerations for a site may also be obtained using a seismic parameter program that has been developed by the United States Geological Survey (USGS) in cooperation with BSSC and FEMA. This program is available on the USGS earthquake hazards web site. The data are interpolated for a specific latitude-longitude or zip code, which the user enters. Output for an entry uses the built-in database to interpolate for the specific site. Caution should be used when using a zip code. In regions with highly variable mapped ground motions, the design parameters within a zip code may vary considerably from the value at the centroid of the zip code area. The code/NEHRP maximum considered earthquake (MCE) output for a site are the two spectral values required for design. The user may also use the program to calculate an MCE response spectrum, with or without site coefficients. Site coefficients can be calculated and included in calculations by simply selecting the site class; the program then calculates the site coefficient.

FIGURES 1613.3.1(1) through 1613.3.1(6). See page 16-74 through 16-81.

- These code figures provide the 5-percent damped spectral response accelerations at 0.2-second period ($S$), as well as at 1-second period ($S_z$) for Site Class B soil profiles. The code has incorporated updated earthquake ground motion maps that reflect the 2008 maps developed by the USGS National Seismic Hazard Mapping Project as well as technical changes adopted for the 2009 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P750). In the NEHRP update process, the title for these maps was revised from "Maximum Considered Earthquake (MCE) Ground Motions" to "Risk-Targeted Earthquake (RTE) Ground Motions."

The seismic hazard maps incorporate new information on earthquake sources and ground motion prediction equations, including the new Next Generation Attenuation (NGA) relations. The ground motion maps further incorporate technical changes that reflect the use of: (1) risk-targeted ground motions; (2) maximum direction ground motions and (3) near-
source 84th percentile ground motions.

Precise design values can be obtained from a USGS web site (http://earthquake.usgs.gov/research/hazmaps/design/index.php) using the longitude and latitude of the building site, obtained from GPS mapping programs or web sites.

1613.3.2 Site class definitions. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 20 of ASCE 7. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines Site Class E or F soils are present at the site.

Each site is to be classified as one of six site classes (A through F), as defined in ASCE 7, based on one of three soil properties measured over the top 100 feet (30 480 mm) of the site. If the top 100 feet (30 480 mm) are not homogeneous, the ASCET provisions address how to determine average properties. Site Class A is hard rock typically found in the eastern United States. Site Class B is softer rock, typical of the western parts of the country. Site Class C, D or E indicates progressively softer soils. From an earthquake-resistance perspective, rock is the best material for most structures to be founded on. Site Class F indicates soil so poor that site-specific geotechnical investigation and dynamic site-response analysis are needed to determine appropriate site coefficients.

The three soil properties forming the basis of site classification are: shear wave velocity, standard penetration resistance or blow count (determined in accordance with ASTM D 1586) and undrained shear strength (determined in accordance with ASTM D 2166 or ASTM D 2850). Site Class A and B designations must be based on shear wave velocity measurements or estimates.

Where soil property measurements to a depth of 100 feet (30 480 mm) are not feasible, the registered design professional performing the geotechnical investigation may estimate appropriate soil properties based on known geologic conditions.

When soil properties are not known in sufficient detail to determine the site class, the default site class is D, unless the building official determines that Site Class E or F soils may exist at the site.

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, $S_m$, and at 1-second period, $S_M$, adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively:

$$S_m = F_p S$$ (Equation 16-37)

$$S_M = F_v S$$ (Equation 16-38)

where:

$F_p = $ Site coefficient defined in Table 1613.3.3(1).

$F_v = $ Site coefficient defined in Table 1613.3.3(2).

$S_m = $ The mapped spectral accelerations for short periods as determined in Section 1613.3.1.

$S_v = $ The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.

Table 1613.3.3(1) defines an acceleration-related or short-period site coefficient, $F_p$, as a function of site class and the mapped spectral response acceleration at 0.2-second period, $S_v$. Table 1613.3.3(2) similarly defines a velocity-related or long-period site coefficient, $F_v$, as a function of site class and the mapped spectral response acceleration at 1-second period, $S_M$. The acceleration-related or short-period site coefficient $F_p$ times $S_v$ is $S_m$, the 5-percent damp soil-modified maximum considered earthquake spectral response acceleration at short periods. The velocity-related or long-period site coefficient $F_v$ times $S_v$ is $S_M$, the 5-percent damp soil-modified maximum considered earthquake spectral response acceleration at 1-second period. Such modification by site coefficients is necessary because the mapped quantities are for Site Class B soils. Softer soils (Site Classes C through E) would typically amplify, and stiffer soils (Site Class A) would deamplify ground motion referenced to Site Class B.

As one would expect, both the short-period site coefficient, $F_p$, and the long-period site coefficient, $F_v$, are equal to unity for the benchmark Site Class B, irrespective of seismicity of the site. For Site Class A, both coefficients are smaller than unity, indicating reduction of benchmark Site Class B ground motion caused by the stiffer soils. For Site Classes C through E, both site coefficients, with the exception of $F_p$ for Site Class E where $S_v \geq 1.00$, are larger than 1, indicating amplification of benchmark Site Class B ground motion on stiffer soils. For the same site class, each site coefficient is typically larger in areas of low seismicity than in areas of high seismicity. The basis of this lies in observations that low-magnitude subsurface rock motion is amplified to a larger extent by overlying softer soils than is high-magnitude rock motion. The site coefficients typically become larger for progressively softer soils. The only exception is provided by the short-period site coefficient in areas of high seismicity ($S_v > 0.75$), which remain unchanged or even decrease as the site class changes from D to E. The basis for this also lies in observations that very soft soils are not capable of amplifying the short-period components of subsurface rock motion: deamplification, in fact, takes place when the subsurface rock motion is high in magnitude.

TABLE 1613.3.3(1). See page 16-72.

$F_p$, the acceleration-related or short-period site coefficient, is defined in this table as a function of site class and the seismicity at the site in the form of the mapped spectral response acceleration at 0.2-second period, $S_v$. 

\[ \text{TABLE 1613.3.3(1)}]
TABLE 1613.3.3(2). See below.

- $F_v$, the velocity-related or long-period site coefficient, is defined in this table as a function of site class and the seismicity at the site in the form of the mapped spectral response acceleration at 1-second period, $S_v$.

### 1613.3.4 Design spectral response acceleration parameters

Five-percent damped design spectral response acceleration at short periods, $S_{DS}$ and at 1-second period, $S_{DS}$ shall be determined from Equations 16-39 and 16-40, respectively:

$$S_{DS} = \frac{2}{3} S_{MS} \quad \text{(Equation 16-39)}$$

$$S_{DI} = \frac{2}{3} S_{MI} \quad \text{(Equation 16-40)}$$

where:

- $S_{MS}$ = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1613.3.3.

- $S_{MI}$ = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1613.3.3.

- The design spectral response acceleration at 0.2-second period, $S_{DS}$, is two-thirds of the $S_{MS}$ value calculated in accordance with Section 1613.3.3. The design spectral response acceleration at 1-second period, $S_{DI}$, is two-thirds of the $S_{MI}$ value calculated in accordance with Section 1613.3.3. Two-thirds is the reciprocal of 1.5; thus, the design ground motion is 1/1.5 times the soil-modified maximum considered earthquake ground motion. This is in recognition of the inherent margin contained in the NEHRP provisions that would make collapse unlikely under 1.5 times the design-level ground motion. The idea is to avoid collapse when a structure is subjected to the soil-modified maximum considered earthquake ground motion.

### 1613.3.5 Determination of seismic design category

Structures classified as Risk Category I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, $S_v$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Structures classified as Risk Category IV that are located where the mapped spectral response acceleration parameter at 1-second period, $S_v$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their risk category and the design spectral response acceleration parameters, $S_{DS}$ and $S_{DI}$, determined in accordance with Section 1613.3.4 or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe seismic design category in accordance with Table 1613.3.3(1) or 1613.3.5(2), irrespective of the fundamental period of vibration of the structure, $T$.

- The seismic design category classification provides a relative scale of earthquake risk to structures. The seismic design category considers not only the seismicity of the site in terms of the mapped spectral response accelerations, but also the site soil profile and the nature of the structure’s risk category (see the step-by-step description of this process under the

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_v \leq 0.25$</td>
<td>$S_v = 0.50$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

- a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, $S_v$.
- b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

### 1613.3.3(2)

VALUES OF SITE COEFFICIENT $F_v$^*  

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_v \leq 0.1$</td>
<td>$S_v = 0.2$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

- a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, $S_v$.
- b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.
definition of "Seismic design category"). It is important to note that there are two tables that are used in order to establish the most restrictive seismic design category classification.

There are two conditions that allow the seismic design category to be determined based upon mapped spectral accelerations, making it unnecessary to go through the process described above. The first is Section 1613.3.1, which identifies areas that have a low seismic risk and are classified as Seismic Design Category A without the need to go through the usual steps. The second instance is an area that is close to a major active fault where $S_r$ is greater than or equal to 0.75g. These areas are deemed to be areas of considerable seismic risk, and the seismic design category is classified as E or F depending on the structure's risk category.

The seismic design category classification is a key criterion in using and understanding the seismic requirements because the analysis method, general design, structural detailing and the structure's component and system design requirements are determined, at least in part, by the seismic design category. Some of the special inspection requirements in Section 1705.11 and the special observation requirements in Section 1704.5 are dependent on the seismic design category classification, as well.

TABLE 1613.3.5(1). See below.

This table defines seismic design category as a function of risk category and the short-period (0.2 second) spectral response acceleration at the site of a structure. As the value of $S_{0S}$ increases, the structure is assigned a higher seismic design category and the earthquake design requirements become more stringent.

<table>
<thead>
<tr>
<th>VALUE OF $S_{0S}$</th>
<th>I or II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{0S} &lt; 0.167g$</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>$0.167g \leq S_{0S} &lt; 0.33g$</td>
<td>B</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>$0.33g \leq S_{0S} &lt; 0.50g$</td>
<td>C</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>$0.50g \leq S_{0S}$</td>
<td>D</td>
<td>D</td>
<td>D</td>
</tr>
</tbody>
</table>

TABLE 1613.3.5(2). See below.

This table defines seismic design category as a function of risk category and the long-period (1-second) spectral response acceleration at the site of a structure. As the value of $S_{0L}$ increases, the structure is assigned a higher seismic design category and the earthquake design requirements become more stringent.

1613.3.5.1 Alternative seismic design category determination. Where $S_I$ is less than 0.75, the seismic design category is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, $T_a$, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 $T_I$ determined in accordance with Section 11.4.5 of ASCE 7.

2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than $T_I$.

3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, $C_s$.

4. The diaphragms are rigid as defined in Section 12.3.1 of ASCE 7 or, for diaphragms that are flexible, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

This section permits the seismic design category of structures meeting the four listed conditions to be based solely on the short-period design spectral coefficient, $S_{0S}$. This is similar to the approach taken in mapping seismic design categories for use in the IRC, as well as for the simplified earthquake analysis procedure.

<table>
<thead>
<tr>
<th>VALUE OF $S_{0L}$</th>
<th>I or II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{0L} &lt; 0.067g$</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>$0.067g \leq S_{0L} &lt; 0.133g$</td>
<td>B</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>$0.133g \leq S_{0L} &lt; 0.20g$</td>
<td>C</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>$0.20g \leq S_{0L}$</td>
<td>D</td>
<td>D</td>
<td>D</td>
</tr>
</tbody>
</table>
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTINUOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(continued)
FIGURE 1813.3.1(1)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B
FIGURE 1613.3.1(2)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(continued)
FIGURE 1613.3.1(2)—continued
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS
FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION
(5% OF CRITICAL DAMPING), SITE CLASS B

2012 INTERNATIONAL BUILDING CODE® COMMENTARY
16-77
**STRUCTURAL DESIGN**

**0.2 Second Spectral Response Acceleration (5% of Critical Damping)**

**1.0 Second Spectral Response Acceleration (5% of Critical Damping)**

**DISCUSSION**

Maps prepared by the United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE). The base is explained in a commentary prepared by USGS and ASCE in the references.

- Ground motion values contained on these maps incorporate:
  - Tectonic effects: all areas equal to 16% or 45 years based upon a generic ground motion history.
- Deterministic: superimposed across large, active faults, which are assigned by 5% for the maximum credible earthquake to the characteristics or magnitude.

As such, the values are different from those on the aforementioned 1994 USGS National Seismic Hazard Maps for Hawaii prepared at http://www.usgs.gov脆弱 shaking maps. Large, more detailed versions of these maps are not provided because as it is recommended that the new mapping technique for USGS with input (http://www.usgs.gov) be developed in order to determine the mapped values for a specific location.

**REFERENCES**


**FIGURE 1613.3.1(3)**

Risk Targeted Maximum Considered Earthquake (MCE) Ground Motion Response Accelerations for Hawaii of 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B
FIGURE 1613.3.1(S)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCEg) GROUND MOTION RESPONSE ACCELERATIONS
FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION
(5% OF CRITICAL DAMPING), SITE CLASS B
STRUCTURAL DESIGN

**REFERENCE**


**FIGURE 1613.3.1(6)**

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

**2012 INTERNATIONAL BUILDING CODE® COMMENTARY**

16-81
1613.3.5.2 Simplified design procedure. Where the alternate simplified design procedure of ASCE 7 is used, the seismic design category shall be determined in accordance with ASCE 7.

A simplified earthquake procedure was first introduced under the 1997 UBC and continued on with minor changes through the first two editions of the code. The 2005 edition of ASCE 7 introduced the first stand-alone version of a simplified earthquake analysis for simple bearing wall or building frame systems that comply with the 12 limitations listed in Section 12.14.1.1 of ASCE 7. For structures that qualify to use this method of analysis, the simplified procedure permits the determination of the seismic design category to be based solely on the short-period design spectral coefficient, $S_{d_{0}}$.

1613.4 Alternatives to ASCE 7. The provisions of Section 1613.4 shall be permitted as alternatives to the relevant provisions of ASCE 7.

This section is intended to provide permitted alternatives to the corresponding ASCE 7 requirements. As such, they should be viewed as options that are available to the design professional in demonstrating compliance with the earthquake provisions.

1613.4.1 Additional seismic force-resisting systems for seismically isolated structures. Add the following exception to the end of Section 17.5.4.2 of ASCE 7:

**Exception:** For isolated structures designed in accordance with this standard, the Structural System Limitations and the Building Height Limitations in Table 12.2-1 for ordinary steel concentrically braced frames (OCBFs) as defined in Chapter 11 and ordinary moment frames (OMFs) as defined in Chapter 11 are permitted to be taken as 160 feet (48 768 mm) for structures assigned to Seismic Design Category D, E or F, provided that the following conditions are satisfied:

1. The value of $R_{u}$ as defined in Chapter 17 is taken as 1.
2. For OMFs and OCBFs, design is in accordance with AISC 341.

The ASCE 7 provisions include requirements for structures that utilize seismic base isolation. It is worth noting that use of seismic isolation is never required by the code. This method is an optional method of design for earthquake loading that has been recognized under legacy model codes, such as the UBC. The modification adds an exception to the ASCE 7 requirements for structural systems above the isolation system. It corrects an oversight that unnecessarily changes prior practice by restricting the use of certain seismic-force-resisting systems that have been used successfully. As modified, it would allow ordinary concentric braced frames and ordinary moment frame systems with heights up to 160 feet (48 768 mm) when the stated conditions are met.

SECTION 1614
ATMOSPHERIC ICE LOADS

1614.1 General. Ice-sensitive structures shall be designed for atmospheric ice loads in accordance with Chapter 10 of ASCE 7.

This section provides charging text in the code in conjunction with technical provisions in ASCE 7 for computing atmospheric ice loads. This section relies on the determination of which structures are ice-sensitive structures in order to determine the need to comply with the applicable provisions of ASCE 7. An "Ice-sensitive structure" is defined in Section 202 and provides a technical basis for determining which structures are ice-sensitive structures.

SECTION 1615
STRUCTURAL INTEGRITY

1615.1 General. High-rise buildings that are assigned to Risk Category III or IV shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1615.3. Bearing wall structures shall comply with the requirements of Section 1615.4.

The requirements in the codes and standards together with the common structural design and construction practices prevalent in the United States have provided the overwhelming majority of structures with adequate levels of reliability and safety. Many buildings simply do not have integrity issues. Low-rise buildings do not represent the same risk as taller, high-rise buildings. The structural integrity provisions reflect this relative risk. By limiting this requirement to Risk Category III and IV buildings that are also high-rise buildings, the provision does not affect buildings that are commonly built and do not exhibit integrity issues.

1615.2 Definitions. The following words and terms are defined in Chapter 2:

Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with these provisions for structural integrity. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

BEARING WALL STRUCTURE.

FRAME STRUCTURE.

1615.3 Frame structures. Frame structures shall comply with the requirements of this section.

These provisions enhance the overall structural integrity and resistance of frame structures by establishing minimum requirements for tying together the primary structural elements.

1615.3.1 Concrete frame structures. Frame structures constructed primarily of reinforced or prestressed concrete, either
1615.3.2 Structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures. Frame structures constructed with a structural steel frame or a frame composed of open web steel joists, joist girders with or without other structural steel elements or a frame composed of composite steel or composite steel joists and reinforced concrete elements shall conform to the requirements of this section.

The provisions contained in this section have been adapted from requirements contained in the ACI 318 standard for many years. By adapting those requirements to structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures, their overall structural integrity is enhanced.

1615.3.2.1 Columns. Each column splice shall have the minimum design strength in tension to transfer the design dead and live load tributary to the column between the splice and the splice or base immediately below.

The additional requirement for the tensile strength of column splices enhances the column’s performance in unforeseen events.

1615.3.2.2 Beams. End connections of all beams and girders shall have a minimum nominal axial tensile strength equal to the required vertical shear strength for allowable stress design (ASD) or two-thirds of the required shear strength for load and resistance factor design (LRFD) but not less than 10 kips (45 kN). For the purpose of this section, the shear force and the axial tensile force need not be considered to act simultaneously.

Exception: Where beams, girders, open web joist and joist girders support a concrete slab or concrete slab on metal deck that is attached to the beam or girder with not less than 3/4-inch-diameter (6.5 mm) headed shear studs, at a spacing of not more than 12 inches (305 mm) on center, averaged over the length of the member, or other attachment having equivalent shear strength, and the slab contains continuous distributed reinforcement in each of two orthogonal directions with an area not less than 0.0015 times the concrete area, the nominal axial tension strength of the end connections shall be permitted to be taken as half the required vertical shear strength for ASD or one-third of the required shear strength for LRFD, but not less than 10 kips (45 kN).

Providing the required tensile strength for all beam and girder connections provides some ability to carry, transfer and/or redistribute load in the event that there is loss of support. The exception allows a reduced tensile strength in the beam and girder connection where a concrete slab is utilized.

1615.4 Bearing wall structures. Bearing wall structures shall have vertical ties in all load-bearing walls and longitudinal ties, transverse ties and perimeter ties at each floor level in accordance with this section and as shown in Figure 1615.4.

These provisions enhance the overall structural integrity and resistance of bearing wall structures by establishing minimum requirements for tying together the primary structural elements.

FIGURE 1615.4. See page 16-84.

This figure illustrates the ties that are required in Sections 1614.4.1 through 1614.4.2.4.

1615.4.1 Concrete wall structures. Precast bearing wall structures constructed solely of reinforced or prestressed concrete, or combinations of these shall conform to the requirements of Sections 7.13, 13.3.8.5 and 16.5 of ACI 318.

This section refers to the ACI 318 requirements that are applicable to concrete wall structures. These provisions establish minimum requirements for tying together the primary structural elements. These ACI 318 structural integrity requirements are already incorporated in the code by the general reference to ACI 318 in Section 1901.2 for the design and construction of structural concrete.

1615.4.2 Other bearing wall structures. Ties in bearing wall structures other than those covered in Section 1615.4.1 shall conform to this section.

The provisions contained in this section have been adapted from requirements contained within the ACI
318 standard for many years. By adapting those requirements to other bearing wall structures, these provisions enhance their overall structural integrity.

1615.4.2.1 Longitudinal ties. Longitudinal ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within or across walls; or connections of continuous framing members to walls. Longitudinal ties shall extend across interior load-bearing walls and shall connect to exterior load-bearing walls and shall be spaced at not greater than 10 feet (3038 mm) on center. Ties shall have a minimum nominal tensile strength, $T_n$, given by Equation 16-41. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_r = w L S \leq \alpha_r S$$

(Equation 16-41)

where:

$L$ = The span of the horizontal element in the direction of the tie, between bearing walls, feet (m).

$w$ = The weight per unit area of the floor or roof in the span being tied to or across the wall, psf (N/m²).

$S$ = The spacing between ties, feet (m).

$\alpha_r$ = A coefficient with a value of 1,500 pounds per foot (2.25 kN/m) for masonry bearing wall structures and a value of 375 pounds per foot (0.6 kN/m) for structures with bearing walls of cold-formed steel light-frame construction.

Requirements for tying together horizontal elements in the longitudinal direction (spanning between bearing walls—noted as “L” in Figure 1615.4) provides for transfer and/or redistribution of loads in the event there is loss of support.

1615.4.2.2 Transverse ties. Transverse ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within or across walls; or connections of continuous framing members to walls. Transverse ties shall be placed no farther apart than the spacing of load-bearing walls. Transverse ties shall have minimum nominal tensile strength $T_n$, given by Equation 16-46. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

Requirements for ties in the transverse direction at bearing walls (noted as “T” in Figure 1615.4) provides for transfer and/or redistribution of loads in the event there is loss of support.

1615.4.2.3 Perimeter ties. Perimeter ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within...
or across walls; or connections of continuous framing members to walls. Ties around the perimeter of each floor and roof shall be located within 4 feet (1219 mm) of the edge and shall provide a nominal strength in tension not less than $T_p$ given by Equation 16-42. For ASD the minimum nominal tensile strength shall be permitted to be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_p = 200ws\beta_f$$  \hspace{1cm} \text{(Equation 16-42)}$

For SI: $T_p = 90.7ws\beta_f$

where:

$w$ = A coefficient with a value of 16,000 pounds (7200 kN) for structures with masonry bearing walls and a value of 4,000 pounds (1300 kN) for structures with bearing walls of cold-formed steel light-frame construction.

- Requirements for tying together horizontal elements at the perimeter in both the longitudinal and transverse directions (noted as "PIL" and "PIT" in Figure 1615.4) provides for transfer and/or redistribution of load in the event there is loss of support.

**1615.4.2.4 Vertical ties.** Vertical ties shall consist of continuous or spliced reinforcing, continuous or spliced members, wall sheathing or other engineered systems. Vertical tension ties shall be provided in bearing walls and shall be continuous over the height of the building. The minimum nominal tensile strength for vertical ties within a bearing wall shall be equal to the weight of the wall within that story plus the weight of the diaphragm tributary to the wall in the story below. No fewer than two ties shall be provided for each wall. The strength of each tie need not exceed 3,000 pounds per foot (450 kN/m) of wall tributary to the tie for walls of masonry construction or 750 pounds per foot (140 kN/m) of wall tributary to the tie for walls of cold-formed steel light-frame construction.

- The additional requirement for continuous vertical ties enhances the performance of bearing walls in unforeseen events (noted as "V" in Figure 1615.4).

**Bibliography**

The following resource materials are referenced in this chapter or are relevant to the subject matter addressed in this chapter.

- ASTM D 2487-06e1, *Standard Classification of Soils for Engineering Purposes Unified Soil Classification*


Chapter 17:
Special Inspections and Tests

General Comments
In this chapter, the code sets minimum quality standards for the acceptance of materials used in building construction. It also establishes requirements for special inspections, structural observations and load testing.

Section 1701 contains the scope statement and general statement for new and used materials.

The terms primarily related to this chapter are listed in Section 1702 and their definitions are provided in Chapter 2.

Section 1703 addresses the approval process and labeling by approved agencies. Special inspections, contractor responsibility and structural observation are specified in Section 1704.

Section 1704 also includes the detailed requirements pertaining to the statement of special inspections.

Section 1705 contains detailed special inspection and verification requirements for various building elements based on the type of construction involved. Included are special inspection and verification requirements for steel, concrete, masonry, wood, soils, deep foundations, wind, seismic, fire resistance, Exterior Insulation and Finish Systems (EIFS) and smoke control. Structural testing and qualification for seismic resistance is also addressed in Section 1705.

The general requirements for determining the design strengths of materials are in Section 1706.

Section 1707 provides for an alternative test procedure in the absence of approved standards.

Provisions for a test load are addressed in Section 1708.

Section 1709 includes requirements for field load testing of a structure.

Preconstruction load testing of materials and methods of construction that are not capable of being designed by an approved analysis is covered by Section 1710.

Section 1711 includes specific material and test standards for joist hangers and concrete and clay tile roof covering.

Chapter 17 provides information regarding the evaluation, inspection and approval process for any material or system proposed for use as a component of a structure. These are general requirements that expand on the requirements of Chapter 1 related to the roles and responsibilities of the building official regarding approval of building components. Additionally, the chapter includes general requirements relating to the roles and responsibilities of the owner, contractor, special inspectors and architects or engineers.

Purpose
This chapter provides procedures and criteria for testing materials or assemblies; labeling materials; systems and assemblies and special inspection and verification of structural assemblies.

SECTION 1701
GENERAL

1701.1 Scope. The provisions of this chapter shall govern the quality, workmanship and requirements for materials covered. Materials of construction and tests shall conform to the applicable standards listed in this code.

This chapter gives provisions for quality, workmanship, testing and labeling of all materials covered within. In general, all construction materials and tests must conform to the standards, or portions thereof, that are referenced in the code. This chapter provides requirements for materials and tests when there are no applicable standards; specific tests and standards are referenced in other chapters of the code. Additionally, this chapter provides basic requirements for labeling construction materials and assemblies, and for special inspection and verification of structural systems and components.

1701.2 New materials. New building materials, equipment, appliances, systems or methods of construction not provided for in this code, and any material of questioned suitability proposed for use in the construction of a building or structure, shall be subjected to the tests prescribed in this chapter and in the approved rules to determine character, quality and limitations of use.

Testing is required to be performed on materials that are not specifically provided for in the code. For example, suppose a manufacturer of a sandwich panel consisting of aluminum skins and a foam plastic core wishes to use this panel as an exterior weather covering. The material does not conform to any of the standards referenced in Chapter 14, so an appropriate test protocol must be developed. The same provision for acceptance of alternative materials is already given in Section 104.11. That section provides a strong, definitive statement for performance requirements for alternative materials, requir-
ing the proposed alternative to be equivalent to that prescribed by the code in quality, strength, effectiveness, durability and safety. Section 1701.2 simply reasserts that alternative materials (new materials) may be used, as long as the performance characteristics and quality can be established.

1701.3 Used materials. The use of second-hand materials that meet the minimum requirements of this code for new materials shall be permitted.

- Materials and assemblies may be reused, provided that they meet the requirements of the code for new materials (see Section 104.9.1 of the code regarding reuse of materials and equipment). Caution should be exercised in approving a used material for reuse. The applicable material standards must be consulted to determine if certain reuses are prohibited and to determine the characteristics of the used material that must be carefully checked before reuse is approved.

    One example is a high-strength structural steel bolt. Reuse of the bolt is restricted by Research Council on Structural Connections (RCSC), Specification for Structural Joints Using ASTM A 325 or A 490 Bolts. Even a piece of structural steel, such as a wide flange, would need to be carefully checked to determine that dimensional tolerances for a new piece of structural steel are met (see ASTM A 6 and A 36).

SECTION 1702 DEFINITIONS

1702.1 Definitions. The following terms are defined in Chapter 2:

- Definitions facilitate the understanding of code provisions and minimize potential confusion. To that end, this section lists definitions of terms associated with special inspections and testing. Note that these definitions are found in Chapter 2. The use and application of defined terms, as well as undefined terms, are set forth in Section 201.

APPROVED AGENCY.

APPROVED FABRICATOR.

CERTIFICATE OF COMPLIANCE.

DESIGNATED SEISMIC SYSTEM.

FABRICATED ITEM.

INSPECTION CERTIFICATE.

INTUMESCENT FIRE-RESISTANT COATINGS.

MAIN WINDFORCE-RESISTING SYSTEM.

Mastic Fire-Resistant Coatings.

SPECIAL INSPECTION.

- Continuous special inspection.

- Periodic special inspection.

SPECIAL INSPECTOR.

SPRAYED FIRE-RESISTANT MATERIALS.

STRUCTURAL OBSERVATION.

SECTION 1703 APPROVALS

1703.1 Approved agency. An approved agency shall provide all information as necessary for the building official to determine that the agency meets the applicable requirements.

- This section specifies the information that an approved agency must provide to the building official to enable him or her to determine if the agency meets the applicable requirements.

1703.1.1 Independence. An approved agency shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.

- As part of the basis for a building official's approval of a particular inspection agency, the agency must demonstrate its objectivity and competence. The judgement of objectivity is linked to the financial and fiduciary independence of the agency. The competence of the agency is judged by its experience and organization, and the experience of its personnel.

    For example, suppose that ACME Agency is the inspection agency employed by Builder's, Inc. for factory-built fireplaces. During an investigation of the agency, it is discovered that ACME and Builder's are subsidiaries of the same parent company, Conglomerate, Inc. The inspection agency and manufacturer clearly have a relationship that is undesirable from the standpoint of independence.

1703.1.2 Equipment. An approved agency shall have adequate equipment to perform required tests. The equipment shall be periodically calibrated.

- As part of judging the ability of a testing or inspection agency, the building official should determine that the agency has the proper equipment to perform the required tests or inspections.

1703.1.3 Personnel. An approved agency shall employ experienced personnel educated in conducting, supervising and evaluating tests and/or inspections.

- The competence of an inspection or testing agency is also based on the experience and background of its personnel. For example, if 10 engineering graduates form an agency, the building official should question whether or not this newly formed agency is sufficiently experienced to perform the tests.

    If the services being provided by the inspection or test agency come within the purview of the professional registration laws of the state in which the building is being constructed, the building official should request evidence that the personnel are qualified to perform the work in accordance with this professional registration law, as well.

2012 INTERNATIONAL BUILDING CODE® COMMENTARY
1703.2 Written approval. Any material, appliance, equipment, system or method of construction meeting the requirements of this code shall be approved in writing after satisfactory completion of the required tests and submission of required test reports.

- In order to have a documented record of the approval and basis for it, including any conditions or limitations, materials and systems must be approved in writing by the building official. The code also requires the approval to be granted within a reasonable period of time, after all documentation has been satisfactorily developed and submitted, so as to avoid any unnecessary delay in completion of construction.

1703.3 Approved record. For any material, appliance, equipment, system or method of construction that has been approved, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the building official's office and shall be open to public inspection at appropriate times.

- Written approvals must be kept on file by the building official, and be available and open to the public. This provides reasonable access to the records on approvals of materials and systems should there be any subsequent investigation or further evaluation.

1703.4 Performance. Specific information consisting of test reports conducted by an approved testing agency in accordance with the appropriate referenced standards, or other such information as necessary, shall be provided for the building official to determine that the material meets the applicable code requirements.

- When conformance to the code is predicated on the performance and quality of materials, the building official must require the submittal of testing reports from an approved agency. In the absence of such reports, the building official must accept specific information and details that prove compliance with the intent of the applicable code requirements.

1703.4.1 Research and investigation. Sufficient technical data shall be submitted to the building official to substantiate the proposed use of any material or assembly. If it is determined that the evidence submitted is satisfactory proof of performance for the use intended, the building official shall approve the use of the material or assembly subject to the requirements of this code. The costs, reports and investigations required under these provisions shall be paid by the applicant.

- This section is usually used in conjunction with Section 104.11 when analysis of any construction material, such as new and innovative materials, is required to determine code compliance. The analysis is based entirely upon technical data. All costs of testing and investigations must be paid by the applicant.

1703.4.2 Research reports. Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in this code, shall consist of valid research reports from approved sources.

- Reports prepared by approved agencies, such as those published by organizations affiliated with model code groups, may be accepted as part of the information needed by the building official to evaluate proposed construction and form the basis for approval. Such reports can supplement the building department resources by eliminating the need for the building official to conduct a detailed analysis on each new product, material or system. It is important that such material be truly objective and credible, and not consist merely of the manufacturer's brochures or similar proprietary information. It is also important to note that when the building official is utilizing research reports in evaluating compliance with the code, such as those issued by organizations affiliated with model code groups, he or she is not mandated to approve these reports just because the code is the legally adopted building code in the jurisdiction. These reports are not code text; they are advisory only and intended for technical reference.

1703.5 Labeling. Where materials or assemblies are required by this code to be labeled, such materials and assemblies shall be labeled by an approved agency in accordance with Section 1703. Products and materials required to be labeled shall be labeled in accordance with the procedures set forth in Sections 1703.5.1 through 1703.5.4.

- This section provides requirements for third-party inspection of a manufacturer of a material or assembly when the code says that the material or assembly must be labeled. The materials or assemblies required to be labeled are given in other chapters of the code, the International Mechanical Code® (IMC®), the International Fire Code® (IFC®) and the International Plumbing Code® (IPC®). Labeling provides a readily available source of information that is useful for field inspection of installed products. The label identifies the product or material and provides other information that can be investigated further if there is any question as to its suitability for the specific installation.

  Some examples are gas appliances, fire doors, prefabricated construction (when the building official does not inspect it), electrical appliances, glass, factory-built fireplaces, plywood and other wood members when used structurally, lumber and foam plastics.

1703.5.1 Testing. An approved agency shall test a representative sample of the product or material being labeled to the relevant standard or standards. The approved agency shall maintain a record of the tests performed. The record shall provide sufficient detail to verify compliance with the test standard.

- As a basis for the allowed use of an agency's label, the agency is required to perform testing on the mate-
SPECIAL INSPECTIONS AND TESTS

Material or product in accordance with the standard referenced by the code. For example, Section 903.1 of the IMC requires that factory-built fireplaces be tested in accordance with the referenced standard UL 127 and states that factory-built fireplaces are required to be listed and labeled by an approved agency.

1703.5.2 Inspection and identification. The approved agency shall periodically perform an inspection, which shall be in-plant if necessary, of the product or material that is to be labeled. The inspection shall verify that the labeled product or material is representative of the product or material tested.

- The approved agency whose label is to be applied to a product must perform periodic inspections. The primary objective of these inspections is to determine that the manufacturer is, indeed, making the same product that was tested. For example, using the factory-built fireplace discussed in the commentary to Section 1703.5.1, if the fire chamber wall in the test was 3/4-inch-thick (9.5 mm) steel, the inspection agency must check to see that this thickness is being used. If the manufacturer has decided to use 1/8-inch (6.4 mm) steel, then the inspection agency would be required to withdraw the use of its label and listing.

1703.5.3 Label information. The label shall contain the manufacturer's or distributor's identification, model number, serial number or definitive information describing the product or material's performance characteristics and approved agency's identification.

- This section lists the information that is required on a label (see Figure 1703.5.3). The purpose is to provide sufficient information for the inspector to verify that the installed product is consistent with what was approved during the plan review process.

![ACME MFG., INC.](image)

Figure 1703.5.3
TYPICAL LABEL INFORMATION

1703.5.4 Method of labeling. Information required to be permanently identified on the product shall be acid etched, sand blasted, ceramic fired, laser etched, embossed or of a type that, once applied, cannot be removed without being destroyed.

- The section requires that permanent labeling be of a nature that cannot be removed and specifies acceptable methods of permanent labeling.

1703.6 Evaluation and follow-up inspection services. Where structural components or other items regulated by this code are not visible for inspection after completion of a prefabricated assembly, the applicant shall submit a report of each prefabricated assembly. The report shall indicate the complete details of the assembly, including a description of the assembly and its components, the basis upon which the assembly is being evaluated, test results and similar information and other data as necessary for the building official to determine conformance to this code. Such a report shall be approved by the building official.

- As an alternative to physical inspection by the building official in the plant or location where prefabricated components are manufactured, such as modular homes, trusses, etc., the building official has the option of accepting an evaluation report from an approved agency detailing such inspections.

1703.6.1 Follow-up inspection. The applicant shall provide for special inspections of fabricated items in accordance with Section 1704.2.5.

- The owner is required to provide special inspections of fabricated assemblies at the fabrication plant in accordance with Section 1704.2.5.

1703.6.2 Test and inspection records. Copies of necessary test and inspection records shall be filed with the building official.

- All testing and inspection records related to a fabricated assembly must be filed with the building official so as to maintain a complete and legal record of the assembly and erection of the building components.

SECTION 1704
SPECIAL INSPECTIONS, CONTRACTOR RESPONSIBILITY AND STRUCTURAL OBSERVATIONS

1704.1 General. This section provides minimum requirements for special inspections, the statement of special inspections, contractor responsibility and structural observations.

- Section 1704 provides minimum requirements for special inspections. It includes in the statement of special inspections, contractor responsibility requirements related to construction of the lateral-force-resisting systems for wind and seismic loads and structural observations.

1704.2 Special inspections. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more approved agen-
cies to perform inspections during construction on the types of work listed under Section 1705. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

1. Special inspections are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.

2. Unless otherwise required by the building official, special inspections are not required for Group U occupancies that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

3. Special inspections are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

- Special inspections provide a means of quality assurance. Structural properties of the concrete or steel that is used in most structures are not usually discernable by a mere visual inspection. Typically, construction materials must be tested and their installation must be monitored in order to provide a finished structure that performs in accordance with the construction documents. Trained specialists that provide these inspections give the building official and engineer an indication that the required structural performance will be achieved. The permit applicant is responsible for hiring the special inspector and must incur all associated costs. According to Section 105.1, the permit applicant may be the owner or authorized agent in connection with the project (see Section 105.1 for further details).

Special inspections do not replace inspections performed by the jurisdiction. Rather, they are intended as an enhancement to those inspections.

Exceptions to the requirement for special inspections include minor work, Group U occupancies accessory to residential occupancies and prescriptive light-frame construction consisting of cold-formed steel or wood framing.

In addition to exempting minor work, Exception 1 refers to "conditions in the jurisdiction" as a possible exception. The primary "condition" envisioned is one in which the jurisdiction has the resources and skills to perform these inspection tasks. This exception should not be interpreted as one that can be invoked by the permit applicant. A local jurisdiction should not be obligated to invoke this exception. The purpose of this exception is merely to allow jurisdictions to perform these inspections to the extent they are capable of doing so, rather than requiring a special inspector.

Exception 2 eliminates the special inspection requirement for Group U occupancies that are accessory to a residential occupancy and where allowed by the building official. Because Group U occupancies could have elements or structural systems that require special inspection, the building official has the authority to require special inspection in such cases.

Exception 3 waives the requirement for special inspections for portions of structures designed and constructed in accordance with the prescriptive provisions of Section 2211.7 for cold-formed steel light-frame structures or of Section 2308 for conventional light-frame wood structures.

1704.2.1 Special inspector qualifications. The special inspector shall provide written documentation to the building official demonstrating his or her competence and relevant experience or training. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code.

The registered design professional in responsible charge and engineers of record involved in the design of the project are permitted to act as the approved agency and their personnel are permitted to act as the special inspector for the work designed by them, provided they qualify as special inspectors.

- This section attempts to standardize special inspector qualifications in an effort to provide for the availability of an adequate pool of qualified and knowledgeable special inspectors. The code requires the special inspector to demonstrate competence to the satisfaction of the building official. There are many certification and training programs for various facets of special inspections that may provide guidance to the building official in making this judgment. The provision allows the registered design professional in responsible charge and engineers of record that designed the project to act as an approved agency and perform special inspections for work designed by them, provided they meet the same qualification requirements for special inspectors.

1704.2.2 Access for special inspection. The construction or work for which special inspection is required shall remain accessible and exposed for special inspection purposes until completion of the required special inspections.

- This section is similar to Section 110.1 in requiring that work remain accessible to the special inspector, until any required inspections have been performed.

1704.2.3 Statement of special inspections. The applicant shall submit a statement of special inspections in accordance with Section 107.1 as a condition for permit issuance. This statement shall be in accordance with Section 1704.3.

Exception: A statement of special inspections is not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

- The applicant must submit for approval a statement of special inspections, in addition to other construction documents, before issuance of the building per-
mit. This section refers to Section 1704.3 for specific details, while Section 1704.3.1 identifies the materials, components and work to be covered in the statement of special inspections.

The exception addresses where the statement of special inspections document is not needed, which is for portions of structures designed and constructed in accordance with the prescriptive provisions of Section 2211.7 for cold formed steel light-frame structures or of Section 2308 for conventional light-frame wood structures.

**1704.2.4 Report requirement.** Special inspectors shall keep records of inspections. The special inspector shall furnish inspection reports to the building official, and to the registered design professional in responsible charge. Reports shall indicate that work inspected was or was not completed in conformance to approved construction documents. Discrepancies shall be brought to the immediate attention of the contractor for correction. If they are not corrected, the discrepancies shall be brought to the attention of the building official and to the registered design professional in responsible charge prior to the completion of that phase of the work.

A final report documenting required special inspections and correction of any discrepancies noted in the inspections shall be submitted at a point in time agreed upon prior to the start of work by the applicant and the building official.

- Records of each inspection must be submitted to the building official so as to compile a complete legal record of the project. These records must include all inspections made, violations and discrepancies. Before a certificate of occupancy is issued, a final report must be submitted indicating that all special inspections have been made and all discrepancies have been resolved or removed in order to show compliance with the applicable code requirements. It is the responsibility of the special inspector to document and submit inspection records to the building official and the registered design professional in responsible charge of the project.

**1704.2.5 Inspection of fabricators.** Where fabrication of structural load-bearing members and assemblies is being performed on the premises of a fabricator’s shop, special inspection of the fabricated items shall be required by this section and as required elsewhere in this code.

- Inspection of in-plant fabrications and the requirements for special in-plant inspections are addressed herein. This section should be used in conjunction with Section 1703.6 relating to evaluation and follow-up inspections.

**1704.2.5.1 Fabrication and implementation procedures.** The special inspector shall verify that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator’s ability to conform to approved construction documents and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator’s scope of work.

**Exception:** Special inspections as required by Section 1704.2.5 shall not be required where the fabricator is approved in accordance with Section 1704.2.5.2.

- The special inspector is required to verify not only that the fabricator complies with the design details and in-house quality control procedures at the plant, but also the plant’s ability to construct/fabricate to the approved drawings, standards and specifications. An example of this would be an inspection of proper placement and rolling of truss-plate connectors at a wood-truss manufacturing plant. Improper procedures could result in the connectors "popping out" or "peeling back" after the truss is concealed and loaded, thus causing structural failure. Special inspections are not necessary where an approved independent agency conducts in-house inspections during fabrication.

**1704.2.5.2 Fabricator approval.** Special inspections required by Section 1705 are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection. Approval shall be based upon review of the fabricator’s written procedural and quality control manuals and periodic auditing of fabrication practices by an approved special inspection agency. At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the building official stating that the work was performed in accordance with the approved construction documents.

- "Approved fabricator" is defined in Section 202 and this section provides the basis for such approval by the building official. If the fabricator is approved by the building official, then its internal quality control procedures are deemed to be sufficient without the need for a special inspection. A certificate of compliance provides the building official with evidence on each project that the work has been performed in accordance with the code and construction documents.

**1704.3 Statement of special inspections.** Where special inspection or testing is required by Section 1705, the registered design professional in responsible charge shall prepare a statement of special inspections in accordance with Section 1704.3.1 for submittal by the applicant in accordance with Section 1704.2.3.

**Exception:** The statement of special inspections is permitted to be prepared by a qualified person approved by the building official for construction not designed by a registered design professional.

- The statement of special inspections is required to be prepared by the registered design professional responsible for the building or structure. This is because the special inspections statement relates directly to the construction documents, which are the
resistance of the registered design professional. The exception states conditions under which the statement of special inspections may be prepared by someone other than the registered design professional. The section outlines the basic content required to be included in the statement of special inspections as well as the specific requirements for seismic and wind resistance, arranging them in logical order. The intent is to document the required inspections and testing in order to foster a systematic approach that helps to achieve the goal of a finished structure that meets or exceeds the minimum performance expectations.

**1704.3.1 Content of statement of special inspections.** The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have special inspection or testing by the building official or by the registered design professional responsible for each portion of the work.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4. Additional requirements for special inspection or testing for seismic or wind resistance as specified in Sections 1705.10, 1705.11 and 1705.12.
5. For each type of special inspection, identification as to whether it will be continuous special inspection or periodic special inspection.

This section details the areas to be addressed in the statement of special inspection. It requires a list of materials and work subject to special inspection, the type and frequency of inspections and whether the inspections are continuous or periodic.

**1704.3.2 Seismic requirements in the statement of special inspections.** Where Section 1705.11 or 1705.12 specifies special inspection, testing or qualification for seismic resistance, the statement of special inspections shall identify the designated seismic systems and seismic force-resisting systems that are subject to special Inspections.

Where special inspection or testing for seismic resistance is required, the statement of special inspections must identify the designated seismic systems and seismic force-resisting systems that are required to have special inspection.

**1704.3.3 Wind requirements in the statement of special inspections.** Where Section 1705.10 specifies special inspection for wind requirements, the statement of special inspections shall identify the main windforce-resisting systems and wind-resisting components subject to special inspection.

Where special inspection for wind resistance is required, the statement of special inspections must identify the main wind-force-resisting systems and components and cladding that are required to have special inspection.

**1704.4 Contractor responsibility.** Each contractor responsible for the construction of a main wind- or seismic force-resisting system, designated seismic system or a wind- or seismic-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the building official and the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain acknowledgement of awareness of the special requirements contained in the statement of special inspection.

The statement of contractor responsibility is required wherever the statement of special inspections includes additional wind- or seismic-resistance inspections. This statement by the contractor is separate from the statement of special inspections. It is the contractor's acknowledgment of the special inspections or testing that are beyond what is typically required.

**1704.5 Structural observations.** Where required by the provisions of Section 1704.5.1 or 1704.5.2, the owner shall employ a registered design professional to perform structural observations as defined in Section 1702.

Prior to the commencement of observations, the structural observer shall submit to the building official a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the building official a written statement that the site visits have been made and identify any reported deficiencies which, to the best of the structural observer's knowledge, have not been resolved.

This section requires that a registered design professional such as an engineer or architect be employed by the owner to provide on-site visits to observe compliance with the structural drawings when required by Section 1704.5.1 or 1704.5.2. Structural observations are required under certain conditions in high-seismic and high-wind locations (see Section 202 for the definition of "Structural observation"). Providing an additional level of inspections beyond those provided by the building inspector or a special inspector. The intent of requiring structural observations by a registered design professional for these structures is to verify that the structural systems are constructed in general conformance with the approved construction documents.

This section provides guidance on the frequency of observations. It requires submittal of a written statement by the structural observer to the building official prior to the commencement of observations identifying the frequency and extent of structural observations. The determination of the frequency of structural observations should be by the structural observer and the owner in consultation with the local building official. The structural observer must also submit a written statement to the building official at the conclusion of the work included in the permit.

**1704.5.1 Structural observations for seismic resistance.** Structural observations shall be provided for those structures
SPECIAL INSPECTIONS AND TESTS

assigned to Seismic Design Category D, E or F where one or more of the following conditions exist:

1. The structure is classified as Risk Category III or IV in accordance with Table 1040.5.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base.
3. The structure is assigned to Seismic Design Category E, is classified as Risk Category I or II in accordance with Table 1040.5, and is greater than two stories above grade plane.
4. When so designated by the registered design professional responsible for the structural design.
5. When such observation is specifically required by the building official.

This section lists the thresholds of seismic risk that necessitate structural observation. Items 1 through 3 are specific conditions that require structural observation, while items 4 and 5 give discretion to the registered design professional or building official to require structural observation in other instances (see commentary, Section 1704.5).

1704.5.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where \( V_{add} \) as determined in accordance with Section 1609.3.1 exceeds 110 mph (49 m/sec), where one or more of the following conditions exist:

1. The structure is classified as Risk Category III or IV in accordance with Table 1604.5.
2. The building height of the structure is greater than 75 feet (22 860 mm).
3. When so designated by the registered design professional responsible for the structural design.
4. When such observation is specifically required by the building official.

This section lists the thresholds of high-wind exposure that necessitate structural observation. Items 1 and 2 are specific conditions that require structural observation, while Items 3 and 4 give discretion to the registered design professional or building official to require structural observation in other instances (see commentary, Section 1704.5).

SECTION 1705
REQUIRED VERIFICATION AND INSPECTION

1705.1 General. Verification and inspection of elements of buildings and structures shall be as required by this section.

The specific elements that require verification by special inspection are covered in Section 1705.

1705.1.1 Special cases. Special inspections shall be required for proposed work that is, in the opinion of the building official, unusual in its nature, such as, but not limited to, the following examples:

1. Construction materials and systems that are alternatives to materials and systems prescribed by this code.

2. Unusual design applications of materials described in this code.

3. Materials and systems required to be installed in accordance with additional manufacturer’s instructions that prescribe requirements not contained in this code or in standards referenced by this code.

This section requires special inspections for proposed work that is unique and not specifically addressed in the code or in standards that are referenced by the code. For example, a designer chooses to utilize a new shear wall anchorage system that is not specifically covered by the code. Because the product is approved as an alternative under Section 104.11, the inspector must rely on installation requirements such as embedment, edge distances, etc., given in approved reports from the manufacturer, provided that the system is previously approved for installation by the building official. Code evaluation reports that require special inspection are based on the requirements in this section.

1705.2 Steel construction. The special inspections for steel elements of buildings and structures shall be as required in this section.

Exception: Special inspection of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator’s ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, and grade for the main stress-carrying elements are capable of being determined. Mill test reports shall be identifiable to the main stress-carrying elements when required by the approved construction documents.

The requirements to be followed by the special inspector for the erection and fabrication of steel elements of building construction are covered in Section 1705.2.

The exception is allowed if the fabrication plant does not utilize any facilities or methods that may alter the physical characteristics or properties of the steel members or components, such as welding, thermal cutting or heating operations. The fabricator would, in any case, need to provide evidence that procedures are used that verify that the proper material specification and grade for the main stress-carrying elements are supplied in accordance with the job specifications and shop drawings.

1705.2.1 Structural steel. Special inspection for structural steel shall be in accordance with the quality assurance inspection requirements of AISC 360.

The 2010 edition of AISC 360-10, AISC Specification for Structural Steel Buildings contains quality assurance and inspection requirements for structural steel buildings. This section references ASIC 360 for spe-
special inspection requirements related to structural steel.

1705.2.2 Steel construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1705.2.2 and this section.

- Special inspection requirements for steel elements other than structural steel, such as cold-formed steel deck and reinforcing steel, are given in Table 1705.2.2.

**TABLE 1705.2.2.** See below.

- The table gives the type of steel elements requiring verification by the special inspector, whether the inspection is continuous or periodic, and the applicable ASTM International (ASTM) material standard or American Welding Society (AWS) welding standard. Note a of the table references Section 1705.11 for any additional special inspections related to seismic resistance.

**1705.2.2.1 Welding.** Welding inspection and welding inspector qualification shall be in accordance with this section.

- This section provides the necessary references that provide guidance on welding inspection for cold-formed steel and reinforcing steel. Welding requirements for structural steel are covered in AISC 360.

**1705.2.2.1.1 Cold-formed steel.** Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

- This section provides a reference to the AWS D1.3 standard that applies to welding inspection and welder inspector qualifications for cold-formed steel floor and roof decks.

**1705.2.2.1.2 Reinforcing steel.** Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

- This section provides the necessary reference to AWS D1.4 and ACI 318 that provides guidance on welding inspection and welder inspector qualifications for reinforcing steel.

**1705.2.2.2 Cold-formed steel trusses spanning 60 feet or greater.** Where a cold-formed steel truss clear span is 60 feet (18 288 mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal package.

- Longer span trusses require additional care during handling and installation due to their size and weight. Special inspection of the bracing provides verification that these critical elements are properly installed in accordance with the approved truss submittal.

**1705.3 Concrete construction.** The special inspections and verifications for concrete construction shall be as required by this section and Table 1705.3.

**Exception:** Special inspections shall not be required for:

1. Isolated spread concrete footings of buildings three stories or less above grade plane that are fully supported on earth or rock.

2. Continuous concrete footings supporting walls of buildings three stories or less above grade plane that are fully supported on earth or rock where:

   2.1. The footings support walls of light-frame construction;

   2.2. The footings are designed in accordance with Table 1809.7; or

<table>
<thead>
<tr>
<th>1. Material verification of cold-formed steel deck:</th>
<th>CONTINUOUS</th>
<th>PERIODIC</th>
<th>REFERENCED STANDARD*</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Identification markings to conform to ASTM standards specified in the approved construction documents.</td>
<td></td>
<td>X</td>
<td>Applicable ASTM material standards</td>
</tr>
<tr>
<td>b. Manufacturer’s certified test reports.</td>
<td></td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

| 2. Inspection of welding: | |
|---|---|---|---|
| a. Cold-formed steel deck: | |
| 1) Floor and roof deck welds. | | X | AWS D1.3 |
| b. Reinforcing steel: | |
| 1) Verification of weldability of reinforcing steel other than ASTM A 706. | | X | |
| 2) Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement. | X | | AWS D1.4 ACI 318: Section 5.5.2 |
| 3) Shear reinforcement. | X | | |
| 4) Other reinforcing steel. | | X | |

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.
2.3. The structural design of the footing is based on a specified compressive strength, $f'_{cu}$, no greater than 2,500 pounds per square inch (psi) (17.2 MPa), regardless of the compressive strength specified in the construction documents or used in the footing construction.

3. Nonstructural concrete slabs supported directly on the ground, including prestressed slabs on grade, where the effective prestress in the concrete is less than 150 psi (1.03 MPa).

4. Concrete foundation walls constructed in accordance with Table 1807.1.6.2.

5. Concrete patios, driveways and sidewalks, on grade.

- This section establishes criteria for special inspections of elements of buildings and structures of concrete construction. Exceptions to the requirements of this section address concrete components that have little or no load-carrying requirements, such as nonstructural slabs on grade, driveways, patios, etc., or footings and foundations that require no reinforcement and carry relatively low loads.

**TABLE 1705.3.** See below.

- Required verifications and inspections during concrete construction operations are listed in Table

<table>
<thead>
<tr>
<th>TABLE 1705.3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION</strong></td>
</tr>
<tr>
<td><strong>VERIFICATION AND INSPECTION</strong></td>
</tr>
<tr>
<td>1. Inspection of reinforcing steel, including prestressing tendons, and placement.</td>
</tr>
<tr>
<td>2. Inspection of reinforcing steel welding in accordance with Table 1705.2.2, Item 2b.</td>
</tr>
<tr>
<td>3. Inspection of anchors cast in concrete where allowable loads have been increased or where strength design is used.</td>
</tr>
<tr>
<td>4. Inspection of anchors post-installed in hardened concrete members*.</td>
</tr>
<tr>
<td>5. Verifying use of required design mix.</td>
</tr>
<tr>
<td>6. At the time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.</td>
</tr>
<tr>
<td>7. Inspection of concrete and shotcrete placement for proper application techniques.</td>
</tr>
<tr>
<td>8. Inspection for maintenance of specified curing temperature and techniques.</td>
</tr>
<tr>
<td>9. Inspection of prestressed concrete:</td>
</tr>
<tr>
<td>a. Application of prestressing forces.</td>
</tr>
<tr>
<td>b. Grouting of bonded prestressing tendons in the seismic force-resisting system.</td>
</tr>
<tr>
<td>10. Erection of precast concrete members.</td>
</tr>
<tr>
<td>11. Verification of in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.</td>
</tr>
<tr>
<td>12. Inspect formwork for shape, location and dimensions of the concrete member being formed.</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspections for seismic resistance.

b. Specific requirements for special inspection shall be included in the research report for the anchor issued by an approved source in accordance with ACI 355.2 or other qualification procedures. Where specific requirements are not provided, special inspection requirements shall be specified by the registered design professional and shall be approved by the building official prior to the commencement of the work.

17-10 2012 INTERNATIONAL BUILDING CODE® COMMENTARY
1705.3. This table provides a complete list of the types of inspections required, whether the inspection is continuous or periodic, and the applicable referenced standards for the placing, curing, prestressing and erection of concrete construction. Note a of the table references Section 1705.11 for any additional special inspections related to seismic resistance.

1705.3.1 Materials. In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials in Chapter 3 of ACI 318, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in Chapter 3 of ACI 318. Weldability of reinforcement, except that which conforms to ASTM A 706, shall be determined in accordance with the requirements of Section 3.5.2 of ACI 318.

- Concrete materials, such as cement, aggregates, admixtures and water, must comply with the standards of Chapter 3 of ACI 318, which regulates materials and addresses specific standards. In the absence of sufficient data or documentation, the building official must require testing in accordance with the standards listed in Chapter 3 of ACI 318.

ASTM A 706 is the standard for reinforcing steel that is weldable, meaning that the chemical composition and manufacturing processes are such that the material is well suited for an acceptable quality of weld. Section 3.5.2 of ACI 318 states that any standard other than ASTM A 706 used for reinforcement material would need to be supplemented for weldability requirements. The intent of this provision is that, where welding of reinforcing steel is required, the steel specified and delivered must be checked for weldability.

1705.4 Masonry construction. Masonry construction shall be inspected and verified in accordance with TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 quality assurance program requirements.

**Exception:** Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, where they are part of structures classified as Risk Category I, II or III in accordance with Section 1604.5.

2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4).

3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

- Because the 2011 edition of the Masonry Standards Joint Committee (MSJC) Code (TMS 402/ACI 530/ASCE 5) contains quality assurance and inspection requirements for masonry structures, this section references TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 MSJC Code and Specification for special inspection requirements related to masonry construction. The three exceptions exempt special inspection for: (1) empirically designed masonry, glass unit masonry or masonry veneer in structures classified as Risk Category I, II or III; (2) prescriptive masonry foundation walls constructed in accordance with the tables in Section 1807.1.6.3; and (3) masonry fireplaces, heaters and chimneys constructed in accordance with the requirements of Chapter 21.

1705.4.1 Empirically designed masonry, glass unit masonry and masonry veneer in Risk Category IV. The minimum special inspection program for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, in structures classified as Risk Category IV, in accordance with Section 1604.5, shall comply with TMS 402/ACI 530/ASCE 5 Level B Quality Assurance.

- This section defines the minimum level of special inspections required for empirically designed masonry, glass unit masonry and masonry veneer in Risk Category IV buildings. The section requires compliance with the Level B Quality Assurance provisions contained in TMS 402/ACI 530/ASCE 5.

1705.4.2 Vertical masonry foundation elements. Special inspection shall be performed in accordance with Section 1705.4 for vertical masonry foundation elements.

- Vertical masonry foundation elements such as masonry foundation walls must meet the special inspection requirements specified for other masonry elements. The section refers to the general provisions for special inspection of masonry structures in Section 1705.4. It should be noted that masonry foundation walls constructed in accordance with the prescriptive provisions and tables in Section 1807.1.6 are specifically exempted from special inspection.

1705.5 Wood construction. Special inspections of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. Special inspections of site-built assemblies shall be in accordance with this section.

- The fabrication process of wood structural elements and assemblies (such as wood trusses) that is being performed on the premises of a fabricator's shop must have special inspection in accordance with Section 1704.2.5. As noted in Section 1704.2.5.2, special inspection is not required in an approved fabricator's shop.

1705.5.1 High-load diaphragms. High-load diaphragms designed in accordance with Section 2306.2 shall be installed with special inspections as indicated in Section 1704.2. The special inspector shall inspect the wood structural panel sheathing to ascertain whether it is of the grade and thickness shown on the approved building plans. Additionally, the special inspector must verify the nominal size of framing members at adjoining panel edges, the nailing or staple diameter and length, the number of fastener lines and that the spacing...
between fasteners in each line and at edge margins agrees with the approved building plans.

- This section requires special inspection of specific portions of high-load diaphragms with multiple rows of fasteners that are designed in accordance with Section 2306.2 (see Table 2306.2(2)). These "high-load" diaphragms have multiple rows of fasteners and are designed to carry higher wind or seismic loads, making it important to have a special inspection during installation.

1705.5.2 Metal-plate-connected wood trusses spanning 60 feet or greater. Where a truss clear span is 60 feet (18 288 mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal package.

- Longer span wood trusses require additional care during handling and installation due to their size and weight. Special inspection of the bracing provides verification that these critical elements are properly installed in accordance with the approved truss submittal.

1705.6 Soils. Special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 1705.6. The approved geotechnical report, and the construction documents prepared by the registered design professionals shall be used to determine compliance. During fill placement, the special inspector shall determine that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report.

Exception: Where Section 1803 does not require reporting of materials and procedures for fill placement, the special inspector shall verify that the in-place dry density of the compacted fill is not less than 90 percent of the maximum dry density at optimum moisture content determined in accordance with ASTM D 1557.

- The load-bearing capacity of the supporting soil has a significant impact on the structural integrity of any building. The amount of compaction and the methods vary depending on the particular design. Use of proper compaction, lift and density, however, is critical to achieving the desired bearing capacity (see Table 1705.6 for the specific inspections that are required). When required by Section 1803, a geotechnical report must be provided, which would list the soil criteria that must be verified. Section 1804.5 clarifies that compacted fill used to support foundations must be in accordance with the criteria of an approved geotechnical report, except for fill depths of 12 inches (305 mm) or less. The exception in this section provides minimum verification requirements of compacted fill when it is not necessary to provide a geotechnical report.

Table 1705.6. See below.

- Tabular formatting of soil inspections clearly conveys the intended requirements for testing and inspection of soils for controlled fill and determination of soil-bearing capacity. The verifications prior to fill placement include verifying that the site preparation meets specified requirements, including proper excavation depth, removal of all deleterious material and any other special requirements that the soils engineer deems necessary for the design (see Items 1, 2 and 5). Verifications of the fill material and the placement operation are treated in Items 3 and 4. Without observing and documenting that the proper material is used and the specified compaction techniques and lifts are employed, the specified load-bearing capacity may not be achieved. A major factor in the design of the fill is the in-place density. This evaluation is needed so that the compaction methods result in adequate soil-bearing capacity.

1705.7 Driven deep foundations. Special inspections shall be performed during installation and testing of driven deep foundation elements as required by Table 1705.7. The approved instruction documents prepared by the registered design professionals, shall be used to determine compliance.

- Table 1705.7 lists the special inspection verifications that are required for installation and testing of driven elements of deep foundations. A geotechnical investigation is required by Section 1803.5.5. The section establishes specific criteria that the geotechnical report should include, which would also serve as the basis for the field verifications by a special inspector.

Table 1705.7. See page 17-13.

- This table provides a concise list of requirements for inspection and verification of materials, testing and

<table>
<thead>
<tr>
<th>TABLE 1705.6 REQUIRED VERIFICATION AND INSPECTION OF SOILS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>VERIFICATION AND INSPECTION TASK</strong></td>
</tr>
<tr>
<td>1. Verify materials below shallow foundations are adequate to achieve the design bearing capacity.</td>
</tr>
<tr>
<td>2. Verify excavations are extended to proper depth and have reached proper material.</td>
</tr>
<tr>
<td>3. Perform classification and testing of compacted fill materials.</td>
</tr>
<tr>
<td>4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of compacted fill.</td>
</tr>
<tr>
<td>5. Prior to placement of compacted fill, observe subgrade and verify that site has been prepared properly.</td>
</tr>
</tbody>
</table>
installation of driven deep foundation elements and indicates whether the inspection is continuous or periodic.

1703.8 Cast-in-place deep foundations. Special inspections shall be performed during installation and testing of cast-in-place deep foundation elements as required by Table 1705.8. The approved geotechnical report, and the construction documents prepared by the registered design professionals, shall be used to determine compliance.

- Table 1705.8 lists the special inspections that are required for cast-in-place elements of deep foundations. A geotechnical investigation is required by Section 1803.5.5. This section establishes specific criteria that the geotechnical report should include, which would also serve as the basis for field verifications by a special inspector.

TABLE 1705.8. See below.

- This table provides a concise list of requirements for inspection and verification of materials, testing and installation of cast-in-place deep foundation elements and indicates whether the inspection is continuous or periodic.

1705.9 Helical pile foundations. Special inspections shall be performed continuously during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque and other pertinent installation data as required by the registered design professional in responsible charge. The approved geotechnical report and the construction documents prepared by the registered design professional shall be used to determine compliance.

- Helical piles are premanufactured and installed by rotating them into the ground (see definition of "Helical pile" in Section 202). As with other types of deep foundation elements, special inspection must be provided to verify that their installation conforms with the approved geotechnical report and construction documents.

<table>
<thead>
<tr>
<th>TABLE 1705.7</th>
<th>REQUIRED VERIFICATION AND INSPECTION OF DRIVEN DEEP FOUNDATION ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERIFICATION AND INSPECTION TASK</td>
<td>CONTINUOUS DURING TASK LISTED</td>
</tr>
<tr>
<td>1. Verify element materials, sizes and lengths comply with the requirements.</td>
<td>X</td>
</tr>
<tr>
<td>2. Determine capacities of test elements and conduct additional load tests, as required.</td>
<td>X</td>
</tr>
<tr>
<td>3. Observe driving operations and maintain complete and accurate records for each element.</td>
<td>X</td>
</tr>
<tr>
<td>4. Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any damage to foundation element.</td>
<td>X</td>
</tr>
<tr>
<td>5. For steel elements, perform additional inspections in accordance with Section 1705.2.</td>
<td>—</td>
</tr>
<tr>
<td>6. For concrete elements and concrete-filled elements, perform additional inspections in accordance with Section 1705.3.</td>
<td>—</td>
</tr>
<tr>
<td>7. For specialty elements, perform additional inspections as determined by the registered design professional in responsible charge.</td>
<td>—</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TABLE 1705.8</th>
<th>REQUIRED VERIFICATION AND INSPECTION OF CAST-IN-PLACE DEEP FOUNDATION ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERIFICATION AND INSPECTION TASK</td>
<td>CONTINUOUS DURING TASK LISTED</td>
</tr>
<tr>
<td>1. Observe drilling operations and maintain complete and accurate records for each element.</td>
<td>X</td>
</tr>
<tr>
<td>2. Verify placement locations and plumbness, confirm element diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end-bearing strata capacity. Record concrete or grout volumes.</td>
<td>X</td>
</tr>
<tr>
<td>3. For concrete elements, perform additional inspections in accordance with Section 1705.3.</td>
<td>—</td>
</tr>
</tbody>
</table>
1705.10 Special inspections for wind resistance. Special inspections itemized in Sections 1705.10.1 through 1705.10.3, unless exemuted by the exceptions to Section 1704.2, are required for buildings and structures constructed in the following areas:

1. In wind Exposure Category B, where $V_{ref}$ is determined in accordance with Section 1609.3.1 is 120 miles per hour (52.8 m/sec) or greater.

2. In wind Exposure Category C or D, where $V_{ref}$ is determined in accordance with Section 1609.3.1 is 110 mph (49 m/sec) or greater.

- This section contains the required additional special inspections in areas that experience higher wind forces. The list of items that need the additional special inspections is related to the general requirement in Section 1704.3.3. This section focuses on the specific areas of concern with respect to wind resistance as detailed in Sections 1705.10.1 for structural wood, Section 1705.10.2 for cold-formed steel and Section 1705.10.3 for components and cladding.

1705.10.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspection is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main windforce-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

- The risk of damage in buildings having main windforce-resisting systems that are constructed of wood warrants additional special inspections. The exception relaxes the special inspection requirement where demands are low, which is reflected by the sheathing fastener spacing of more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspection is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

- The risk of damage in buildings having main windforce-resisting systems that are constructed of cold-formed steel warrants additional special inspections. The exception relaxes the special inspection requirement where demands are low, which is reflected by the sheathing fastener spacing of more than 4 inches (102 mm) on center, or where gypsum board or fiberboard sheathing is used.

1705.10.3 Wind-resisting components. Periodic special inspection is required for the following systems and components:

1. Roof cladding.
2. Wall cladding.

- The hazard addressed by this section is the cladding on buildings and structures in areas expected to experience higher wind forces. Damage to buildings due to high wind forces often begins with a failure of the cladding system, which in turn exposes the main windforce-resisting system to forces that this system is typically not designed to withstand, as well as the effects of wind-driven rain. Wind-driven rain damage occurs to building interiors when roof coverings are blown off, windows are broken or other parts of the structure fail. These are major causes of insurance claims. While such damage is not necessarily life-threatening to the occupants, it does result in costly repairs and renovations. Estimates from Hurricane Andrew, for example, indicate that water damage was responsible for roughly 60 percent of the insured losses.

1705.11 Special inspections for seismic resistance. Special inspections itemized in Sections 1705.11.1 through 1705.11.8, unless exemuted by the exceptions of Section 1704.2, are required for the following:

1. The seismic force-resisting systems in structures assigned to Seismic Design Category C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.

2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F in accordance with Section 1705.11.4.

3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.

4. Storage racks in structures assigned to Seismic Design Category D, E or F in accordance with Section 1705.11.7.

5. Seismic isolation systems in accordance with Section 1705.11.8.

Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 are not required for struc-
structures designed and constructed in accordance with one of the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, $S_{DP}$ as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).

2. The seismic force-resisting system of the structure consists of reinforced concrete; the design spectral response acceleration at short periods, $S_{DP}$ as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).

3. The structure is a detached one- or two-family dwelling not exceeding two stories above grade plane and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
   
   3.1. Torsional or extreme torsional irregularity.
   3.2. Nonparallel systems irregularity.
   3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
   3.4. Discontinuity in lateral strength-weak story irregularity.

The added special inspection requirements for seismic resistance in this section are an important consideration in carrying out the intent of the seismic provisions of the code. A certain amount of inelastic behavior is inherent in a building’s response to the design earthquake. Strong ground shaking caused by earthquakes tends to expose any underlying flaws in a building’s construction or design. Thus, the code specifies additional special inspections of various structural and nonstructural components for seismic resistance in order to provide further verification that these portions of the finished structure are constructed in accordance with the construction documents. It is imperative that the registered design professional who designs systems that are critical to the earthquake performance also identifies them and specifies the necessary inspection and testing.

Item 1 states the seismic risk threshold for seismic-force-resisting systems base on seismic design category. The inspections of items described in Section 1705.11.1 through 1705.11.3 would apply to these structures. Item 2 covers designated seismic systems that are nonstructural components and systems that are assigned an importance factor greater than 1. The only inspections that apply specifically to designated seismic systems are given in Section 1705.11.4, but other requirements, such as those in Section 1705.11.6 for mechanical and electrical components in general, would also be applicable where the component is given an importance factor greater than 1. Item 3 covers special inspections for architectural components in Section 1705.11.5 and for electrical and mechanical components in Section 1705.11.6. Note that the applicable Seismic Design Categories (SDCs) are not specified in Item 3, so the specific SDCs for the various items covered in Sections 1705.11.5 and 1705.11.6 must be followed.

Periodic special inspection is required for storage racks 8 feet or greater in height in buildings in SDC D, E or F as required by Section 0405.11.7.

Section 1705.11.8 requires periodic special inspection during fabrication and installation of isolator units and energy dissipation devices for seismic isolation systems.

Exceptions 1 and 2 apply to light-frame structures with a height no greater than 35 feet and concrete or masonry structures not more than 25 feet in height that have a relatively low seismic risk (where $S_{DS}$ does not exceed 0.50g).

Exception 3 applies to detached one- or two-family dwellings not exceeding two stories above grade plane without significant structural irregularities. The exception is limited to those structures that do not have any of the following irregularities: torsional irregularity; extreme torsional irregularity; nonparallel systems; stiffness irregularity (soft story); stiffness irregularity (extreme soft story) or discontinuity in capacity (weak story). It is important to emphasize that this exception is for the exemption from additional special inspections, not for the design of the structure in accordance with the structural the requirements of the code.

1705.11.1 Structural steel. Special inspection for structural steel shall be in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, $R$, of 3 or less, excluding cantilever column systems.

Section 1705.11 requires the special inspection of seismic-force-resisting systems of structures classified as Seismic Design Category C, D, E or F. This section specifically requires special inspection of the structural steel system in accordance with AISC 341 with the one exception. Also note that Section 2205.2.2 requires structural steel seismic-force-resisting systems to be designed in accordance with AISC 341 for structures in Seismic Design Category D, E or F.

Section 1705.11 makes a general reference to Seismic Design Category C and higher for seismic-force-resisting systems because many of the structural systems that are required utilize special seismic detailing; however, the general requirement does not reflect the allowance in Section 2205.2.1 for steel systems that are not specifically detailed for seismic resistance. The exception recognizes that buildings classified as Seismic Design Category C may use a seismic system that is designed using a response
coefficient, $R = 3$. This recognizes the inherent ductility of these structures that are permitted to be constructed in accordance with AISC 360 (i.e., not detailed in accordance with the provisions of AISC 341). As these construction details and connections are the same as would be used in any steel buildings designed according to AISC 360, no additional inspection or testing is required beyond that required for typical steel buildings in Section 1705.2.1.

1705.11.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: Special inspection is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Continuous special inspection of field gluing operations of structural wood is required, while special inspection of fastenings of components within the seismic-force-resisting system is required on a periodic basis. Since special inspection should only be necessary for complex, highly loaded systems, an exception waives the special inspection of fastening in shear walls and diaphragms that are lightly loaded. Lower design loads mean these elements will have less stringent requirements for bolting, anchoring and fastening. The building official who is already inspecting the sheathing will also inspect fastening in these instances. Rather than being based on the component's design shear, the exception is based on the sheathing fastener spacing as a more practical threshold. All things being equal, as the fastener spacing decreases the component's design load is higher and the tighter spacing increases the potential for splitting framing members. The latter concern is a quality issue associated primarily with nailing.

1705.11.3 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the seismic force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspection is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

Periodic special inspection is required for cold-formed steel framing and its fastening. The exception relaxes the special inspection requirement where demands are low, which is reflected by the sheathing fastener spacing of more than 4 inches (102 mm) on center, or where gypsum board or fiberboard sheathing is used.

1705.11.4 Designated seismic systems. The special inspector shall examine designated seismic systems requiring seismic qualification in accordance with Section 1705.12.3 and verify that the label, anchorage or mounting conforms to the certificate of compliance.

Exception: For elements of the designated seismic system as defined in Section 11.2 of ASCE 7, the special inspector is required to verify that the component label, anchorage or mounting conforms to the certificate of compliance.

1705.11.5 Architectural components. Periodic special inspection is required during the erection and fastening of exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in structures assigned to Seismic Design Category D, E or F.

Exceptions:

1. Special inspection is not required for exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer 30 feet (9144 mm) or less in height above grade or walking surface.
2. Special inspection is not required for exterior cladding and interior and exterior veneer weighing 5 psf (24.5 N/m²) or less.
3. Special inspection is not required for interior nonbearing walls weighing 15 psf (73.5 N/m²) or less.

Although this section is titled "Architectural components," the requirements refer specifically to exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer. It follows that architectural components other than exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer are not subject to these inspections. It applies to nonbearing walls, which are considered nonstructural components as opposed to bearing walls, which are structural elements, and are therefore subject to the special inspections required for the material used to construct the wall. Periodic special inspection of the installation of the listed architectural components verifies that the intent of the design is carried out in the field. The listed components are often supported at different levels of a structure, meaning the component and connection design must consider relative movements. Where connections are designed to allow for relative displacements, verifying that the installation is in accordance with the construction documents is a prudent measure. The special inspector should verify that the method of
anchoring or fastening and the number, spacing and types of fasteners actually used conform with the approved construction documents for the component installed.

Exception 1 exempts construction that is not more than 30 feet (9144 mm) above grade or above a walking surface. Exception 2 exempts cladding and veneer weighing 5 psf (24.5 N/m²) or less from special inspection. Exception 3 exempts interior nonbearing walls, but not exterior nonbearing walls, weighing 15 psf (73.5 N/m²) or less from special inspection. Restated, this section requires periodic special inspection for the erection and fastening of exterior nonbearing walls, cladding and veneer weighing more than 5 psf (24.5 N/m²) and interior nonbearing walls weighing more than 15 psf (73.5 N/m²) when any of these components are installed more than 30 feet (9144 mm) above grade or a walking surface.

1705.11.5.1 Access floors. Periodic special inspection is required for the anchorage of access floors in structures assigned to Seismic Design Category D, E or F.

- Access floors consist of a system of panels and supports that create a raised floor above the actual structural floor system. By raising the floor, a space is created in between the raised floor and the structural floor where various components like wiring for power, voice, and data can be routed. This space has also become increasingly valuable for heating, ventilation, and air-conditioning (HVAC) distribution either as a plenum space or with defined ductwork. Because failure of the floor system can pose a threat to the occupants in high seismic areas, anchorage of access floors require periodic special inspection in buildings in Seismic Design Category D, E and F.

1705.11.6 Mechanical and electrical components. Special inspection for mechanical and electrical components shall be as follows:

1. Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems in structures assigned to Seismic Design Category C, D, E or F;
2. Periodic special inspection is required during the anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F;
3. Periodic special inspection is required during the installation and anchorage of piping systems designed to carry hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F;
4. Periodic special inspection is required during the installation and anchorage of ductwork designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F; and
5. Periodic special inspection is required during the installation and anchorage of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the construction documents require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.

- It is anticipated that the minimum requirements for mechanical and electrical components will be complied with when the special inspector is satisfied that the method of anchoring or fastening, and the number, spacing and types of fasteners actually used, conforms to the approved construction documents for the components installed. It is noted that such special inspection requirements are for selected electrical, lighting, piping and ductwork components in Seismic Design Category C, D, E or F. Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems, installation and anchorage of piping systems designed to carry hazardous materials and their mechanical units, and installation and anchorage of ductwork designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F. Periodic special inspection is required during the anchorage for other electrical equipment in structures assigned to Seismic Design Category E or F.

- Item 5 requires the inspection of vibration isolation systems where an optional lower clearance is specified by the design. Typically, vibration-isolated mechanical equipment requires an increased seismic design force. Note b of Table 13.6-1 of ASCE 7 provides an option to reduce this seismic design force on vibration-isolated components and systems having a clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint. To confirm that the design intent is carried out in the field, isolated equipment installations that utilize a reduced clearance are subject to special inspection.

1705.11.7 Storage racks. Periodic special inspection is required during the anchorage of storage racks 8 feet (2438 mm) or greater in height in structures assigned to Seismic Design Category D, E or F.

- Tall storage racks such as those found in large building supply stores pose a threat to the public in high seismic areas. Because anchorage is the most critical element, anchorage of storage racks greater than or equal to 8 feet (2438 mm) in height in structures assigned to Seismic Design Category D, E or F require periodic special inspection.

1705.11.8 Seismic isolation systems. Periodic special inspection shall be provided for seismic isolation systems during the fabrication and installation of isolator units and energy dissipation devices.

- Seismic isolation units and energy dissipation devices are required to have periodic special inspection during fabrication and installation. The appropriate threshold for requiring such inspections would be that listed for seismic-force-resisting systems in Item 1 of Section 1705.11.
1705.12 Testing and qualification for seismic resistance.
The testing and qualification specified in Sections 1705.12.1 through 1705.12.4, unless exempted from special inspections by the exceptions of Section 1704.2 are required as follows:

1. The seismic force-resisting systems in structures assigned to Seismic Design Category C, D, E or F shall meet the requirements of Sections 1705.12.1 and 1705.12.2, as applicable.

2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F and subject to the certification requirements of ASCE 7 Section 13.2.2 shall comply with Section 1705.12.3.

3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F and where the requirements of ASCE 7 Section 13.2.1 are met by submittal of manufacturer’s certification, in accordance with Item 2 herein, shall comply with Section 1705.12.3.

4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1705.12.4.

This section specifies when material seismic-resistance tests for seismic-force-resisting systems and designated seismic systems are required. These requirements supplement the test requirements contained in the referenced standards given in other sections of the code. The seismic provisions of the code are frequently based on assumed material behavior. Material tests are key to verifying the quality of material that is used for certain seismic-resistant construction.

Some inelastic behavior can be anticipated in a building’s response to the design earthquake. Strong ground shaking caused by earthquakes tends to expose any underlying flaws in a building’s construction or design. Thus, the code specifies testing of some structural and nonstructural components as additional verification that the seismic resistance provided in the finished structure meets the intent of the design. It is imperative that the professional who designs systems that are critical to the earthquake performance also identifies them and specifies the necessary testing in the statement of special inspections.

Item 1 states the seismic risk threshold for seismic-force-resisting systems. The testing of items in Sections 1705.12.1 and 1705.12.2 would apply to these structures. Item 2 covers designated seismic systems that are nonstructural components and systems that are assigned an importance factor greater than 1. The testing applicable to designated seismic systems is given in Section 1705.12.3. Item 3 requires testing for architectural, electrical and mechanical components where the general design requirements of ASCE 7 Section 13.2.1. Item 2 for manufacturer’s certification are satisfied by testing. Testing of seismic isolation systems is covered in Section 1705.12.4 which references Section 17.8 of ASCE 7.

1705.12.1 Concrete reinforcement. Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames, special structural walls and coupling beams connecting special structural walls, in structures assigned to Seismic Design Category B, C, D, E or F, the reinforcement shall comply with Section 21.1.5.2 of ACI 318. Certified mill test reports shall be provided for each shipment of such reinforcement. Where reinforcement complying with ASTM A 615 is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

Certified material test reports are required for rebar. When ASTM A 615 is used in special moment frames and shear walls, the testing requirements of Section 21.1.5.2 of ACI 318 must be used. Where ASTM A 615 rebar is to be welded, a material properties report must be provided in accordance with ACI 318 Section 3.5.2 to determine that the weldability of the steel conforms to AWS D1.4.

1705.12.2 Structural steel. Testing for structural steel shall be in accordance with the quality assurance requirements of AISC 341.

Exception: Testing for structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

Structural steel must be tested as required by AISC 341. Appendix Q of AISC 341 provides the code user with the minimum acceptable requirements for a quality assurance plan that applies to the construction of welded joints, bolted joints and other details in the seismic-force-resisting system. Where appropriate, the appendix references AWS D1.1 for specific acceptance criteria.

Section 1705.12 makes a general reference to Seismic Design Category C and higher for seismic-force-resisting systems because many of the structural systems that are required utilize special detailing; however, the general requirement does not reflect the allowance in Section 2205.2.1 for steel systems not specifically detailed for seismic resistance.

The exception recognizes that buildings classified as Seismic Design Category C may employ a seismic system that is designed using a response coefficient, R = 3. This recognizes the inherent ductility of these structures that are permitted to be constructed in accordance with AISC 360, and not be detailed in accordance with the provisions of AISC 341. As these construction details and connections are the same as would be used in any steel building designed according to AISC 360, no additional inspection or testing is required beyond that required for typical steel buildings in Section 1705.2.1.

1705.12.3 Seismic certification of nonstructural components. The registered design professional shall specify on the construction documents the requirements for certification by
Special Inspections and Tests

1705.12.4 Seismic isolation systems. Seismic isolation systems shall be tested in accordance with Section 17.8 of ASCE 7.

The referenced section of ASCE 7 contains detailed provisions for isolation system testing. This testing provides effective stiffness and damping values to be used in the design of a seismically isolated structure. A minimum of two full-size specimens must be tested for each proposed type and size of isolator. These prototypes are not to be used in the construction, unless approved by both the registered design professional and building official.

1705.13 Sprayed fire-resistant materials. Special inspections for sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural members shall be in accordance with Sections 1705.13.1 through 1705.13.6. Special inspections shall be based on the fire-resistance design as designated in the approved construction documents. The tests set forth in this section shall be based on samplings from specific floor, roof and wall assemblies and structural members. Special inspections shall be performed after the rough installation of electrical, automatic sprinkler systems, mechanical and plumbing systems and suspension systems for ceilings, where applicable.

Special inspections for sprayed fire-resistant materials shall be performed to ensure that the materials meet the design requirements. The application must be in accordance with the terms and conditions of the listing and the manufacturer's instructions. The special inspection of SFRM verifies that the requirements for thickness, density and bond strength that are specified in the design have been satisfied by the actual installation. These inspections are to be performed after the rough installation of mechanical, electrical, plumbing, automatic sprinkler systems and ceiling suspension systems.

1705.13.1 Physical and visual tests. Special inspections shall include the following tests and observations to demonstrate compliance with the listing and the fire-resistance rating:

1. Condition of substrates.
2. Thickness of application.
3. Density in pounds per cubic foot (kg/m³).
5. Condition of finished application.

To verify that an SFRM performs as intended, certain conditions are required to be met. The verifications include: substrate conditions; thickness and density of material, as well as the condition of the finished application. Section 704.13 states the substrate condition and finished condition requirements.

1705.13.2 Structural member surface conditions. The surfaces shall be prepared in accordance with the approved fire-resistance design and the written instructions of approved manufacturers. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

The integrity of an SFRM system depends on the conditions of the surface of the steel member to which it is to be applied. The system must be fully adhered to the surface for proper performance, in accordance with design values. See Section 704.13.3.1 for requirements related to the substrate surface conditions.

1705.13.3 Application. The substrate shall have a minimum ambient temperature before and after application as specified in the written instructions of approved manufacturers. The area for application shall be ventilated during and after application as required by the written instructions of approved manufacturers.

During application of SFRMs, and immediately thereafter during cure of the material, several items must be controlled, including the ambient temperature during the application and temperature of the substrate and the SFRMs. Temperature control is important to determine that the necessary chemical reactions needed to make a particular material bond to the steel surfaces and hold together do, in fact, happen. The minimum or maximum temperatures necessary for proper bond and cure depend on the specific type of material (see Section 704.13.4 for more information). The scope of the special inspection also
includes verification of the proper ventilation during application, as well as for curing.

1705.13.4 Thickness. No more than 10 percent of the thickness measurements of the sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural members shall be less than the thickness required by the approved fire-resistance design, but in no case less than the minimum allowable thickness required by Section 1705.13.4.1.

- For the system to provide the required design fire-resistance rating, it must be applied at the appropriate thickness. This section establishes an acceptable percentage of thickness readings that can fall below the specified design value, provided none of these readings are less than the minimum established in Section 1705.13.4.1. These limitations on the thickness measurements are intended to provide a high confidence level that the installed material meets, or exceeds, the design requirements.

1705.13.4.1 Minimum allowable thickness. For design thicknesses 1 inch (25 mm) or greater, the minimum allowable individual thickness shall be the design thickness minus 1/4 inch (6.4 mm). For design thicknesses less than 1 inch (25 mm), the minimum allowable individual thickness shall be the design thickness minus 25 percent. Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Sections 1705.13.4.2 and 1705.13.4.3.

- This requirement prevents the combination of very thin readings with thicker readings in order to show compliance. The required sampling provided for floor, roof and wall assemblies in Section 1705.13.4.2 and structural members in Section 1705.13.4.3 is based on ASTM E 605 with some modifications. This standard also provides testing methods that are commonly used by the industry.

1705.13.4.2 Floor, roof and wall assemblies. The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, making not less than four measurements for each 1,000 square feet (93 m²) of the sprayed area, or portion thereof, in each story.

- Sampling of an SFRM for membrane components (floors, roofs or walls) is based on the square footage of the components. The number of samples is increased in the code by using a sampling area of every 1,000 square feet (93 m²) rather than 10,000 square feet (929 m²) as is specified in ASTM E 605. This is intended to provide a higher level of confidence in the performance of the installed assembly.

1705.13.4.3 Cellular decks. Thickness measurements shall be selected from a square area, 12 inches by 12 inches (305 mm by 305 mm) in size. A minimum of four measurements shall be made, located symmetrically within the square area.

- This section is consistent with the procedure in ASTM E 605 for flat decks.

1705.13.4.4 Fluted decks. Thickness measurements shall be selected from a square area, 12 inches by 12 inches (305 mm by 305 mm) in size. A minimum of four measurements shall be made, located symmetrically within the square area, including one each of the following: valley, crest and sides. The average of the measurements shall be reported.

- This section is consistent with the procedure in ASTM E 605 for fluted decks.

1705.13.4.5 Structural members. The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

- Sampling of the SFRM for structural elements is based on the square footage of each floor. Sample size and number of elements represented are based on ASTM E 605.

1705.13.4.6 Beams and girders. At beams and girders thickness measurements shall be made at nine locations around the beam or girder at each end of a 12-inch (305 mm) length.

- This section is consistent with the procedure in ASTM E 605 for beams.

1705.13.4.7 Joists and trusses. At joists and trusses, thickness measurements shall be made at seven locations around the joist or truss at each end of a 12-inch (305 mm) length.

- This section is consistent with the procedure in ASTM E 605 for joists.

1705.13.4.8 Wide-flanged columns. At wide-flanged columns, thickness measurements shall be made at 12 locations around the column at each end of a 12-inch (305 mm) length.

- This section is consistent with the procedure in ASTM E 605 for columns.

1705.13.4.9 Hollow structural section and pipe columns. At hollow structural section and pipe columns, thickness measurements shall be made at a minimum of four locations around the column at each end of a 12-inch (305 mm) length.

- This section provides guidance for the thickness sampling of hollow structural sections that is not provided in ASTM E 605.

1705.13.5 Density. The density of the sprayed fire-resistant material shall not be less than the density specified in the approved fire-resistance design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605. The test samples for determining the density of the sprayed fire-resistant materials shall be selected as follows:

1. From each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m²) or portion thereof of the sprayed area in each story.

2. From beams, girders, trusses and columns at the rate of not less than one sample for each type of structural
member for each 2,500 square feet (232 m²) of floor area or portion thereof in each story.

- The density of an SFRM will have an impact on the fire-resistance rating of the system; therefore, it is important that the density of the material be measured to verify that the product is as designed. The sampling requirements differ from those for thickness measurements. The required method of determining density is provided in ASTM E 605, but the sample size is based on a sampling area of every 2,500 square feet (232 m²) rather than 10,000 square feet (929 m²), as is specified in ASTM E 605. This is intended to provide a higher level of confidence in the performance of the finished assembly.

1705.13.6 Bond strength. The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to floor, roof and wall assemblies and structural members shall not be less than 150 pounds per square foot (psf) (7.18 kN/m²). The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with Sections 1705.13.6.1 through 1705.13.6.3.

- The adhesion of a sprayed-on material is critical to its performance. This is the key factor in minimizing the chances of the material becoming dislodged. A minimum cohesive/adhesive bond strength of 150 pounds per square foot (psf) (7.18 kN/m²) is required in this section based on the American Institute of Architects (AIA) Master Specification and the recommendations of the General Services Administration (GSA) for durability and serviceability of the material. While this minimum bond strength may be generally suitable, note that Section 403.2.4 provides more stringent requirements that apply to high-rise buildings, where the consequences of dislodged materials can be much greater.

1705.13.6.1 Floor, roof and wall assemblies. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m²) of the sprayed area, or portion thereof, in each story.

- The sampling rate for bond in this section matches the sampling rate for determining density.

1705.13.6.2 Structural members. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, trusses, columns and other structural members at the rate of not less than one sample for each type of structural member for each 2,500 square feet (232 m²) of floor area or portion thereof in each story.

- The bond strength sampling rate in this section is the same as that indicated in Section 1705.13.5 for determining density.

1705.13.6.3 Primer, paint and encapsulant bond tests. Bond tests to qualify a primer, paint or encapsulant shall be conducted when the sprayed fire-resistant material is applied to a primed, painted or encapsulated surface for which acceptable bond-strength performance between these coatings and the fire-resistant material has not been determined. A bonding agent approved by the SFRM manufacturer shall be applied to a primed, painted or encapsulated surface where the bond strengths are found to be less than required values.

- The in-place adhesion of SFRM can be reduced by a factor of 10 when applied over certain primers as opposed to the adhesion obtained by the rated material applied on bare, clean steel. Where the listing does not consider application over such materials, then its bond strength must be determined as described in this section. It is necessary to apply a bonding agent when the bond strength is less than required due to the effect of an encapsulated, painted or primed surface. Also see Section 704.13.3.2 for additional conditions on application of the SFRM.

1705.14 Mastic and intumescent fire-resistant coatings. Special inspections for mastic and intumescent fire-resistant coatings applied to structural elements and decks shall be in accordance with AWCI 12-B. Special inspections shall be based on the fire-resistance design as designated in the approved construction documents.

- Special inspection of mastic and intumescent fire-resistant coatings is justified because these products are often complex systems that require special expertise from applicators and quality assurance personnel. It is essential to confirm that their installation is in accordance with the manufacturer’s instructions and the terms of their listing so that they will perform as expected.

1705.15 Exterior insulation and finish systems (EIFS). Special inspections shall be required for all EIFS applications.

Exceptions:
1. Special inspections shall not be required for EIFS applications installed over a water-resistive barrier with a means of draining moisture to the exterior.
2. Special inspections shall not be required for EIFS applications installed over masonry or concrete walls.

- Special inspections are required for all EIFS installations except for the two exceptions in this section. Exception 1 recognizes that EIFS, which are installed over a water-resistive barrier and incorporate flashings at penetrations and terminations, and a means of drainage to the exterior, afford a built-in redundancy to water penetration that makes the need for special inspections less critical.

Exception 2 recognizes that concrete and masonry substrates are relatively durable and the exposure to moisture in wall conditions does not necessarily have a detrimental effect on these materials.

1705.15.1 Water-resistive barrier coating. A water-resistive barrier coating complying with ASTM E 2570 requires
special inspection of the water-resistive barrier coating when installed over a sheathing substrate.

- Where an EIFS is utilized in residential occupancies of Type V construction, an EIFS with drainage is required by Chapter 14. This specific type of system incorporates a means of drainage applied over a water-resistive barrier and Chapter 14 allows a water-resistive barrier coating complying with ASTM E 2570 as an option. This section requires special inspection of that barrier coating.

1705.16 Fire-resistant penetrations and joints. In high-rise buildings or in buildings assigned to Risk Category III or IV in accordance with Section 1604.5, special inspections for through-penetrations, membrane penetration firestops, fire-resistant joint systems, and perimeter fire barrier systems that are tested and listed in accordance with Sections 714.3.1.2, 714.4.1.2, 715.3 and 715.4 shall be in accordance with Section 1705.16.1 or 1705.16.2.

- Through-penetration and membrane-penetration firestop systems, as well as fire-resistant joint systems and perimeter fire barrier systems, are critical to maintaining the fire-resistive integrity of fire-resistance-rated construction, including fire walls, fire barriers, fire partitions, smoke barriers and horizontal assemblies. The proper selection and installation of such systems must be in compliance with the code and/or appropriate listing. With thousands of listed firestop and joint systems available, each with variations that multiplies possible systems for a building exponentially, the selection of the correct system is not a generic process. Where such systems are used in two types of buildings considered as "high risk," it is mandatory that they be included as a part of the special inspection process. Such "high risk" buildings are identified as:
  
  - Buildings assigned to Risk Category III or IV in accordance with Section 1604.5 and
  - High-rise buildings.

Although the proper application of firestop and joint system requirements is very important in all types and sizes of buildings, the requirement for special inspection is limited to specific building types that represent a substantial hazard to human life in the event of a system failure or that are considered to be essential facilities. Inspection to ASTM E 2174 for penetration firestop systems and ASTM E 2393 for fire-resistant joint systems brings an increased level of review to this important discipline.

1705.16.1 Penetration firestops. Inspections of penetration firestop systems that are tested and listed in accordance with Sections 714.3.1.2 and 714.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

- A primary method of addressing a penetration of a fire-resistance-rated wall assembly is through the use of an approved firestop system installed in accordance with ASTM E 814 or UL 1479. The system must have an F rating that is not less than the fire-resistance rating of the wall being penetrated. It is critical that the firestop system be appropriate for the penetration being protected. The choice of firestop systems varies based upon the size and material of the penetrating item, as well as the construction materials and fire-resistance rating of the wall being penetrated. Special inspection of the firestop system is intended to verify that the appropriate system has been specified and the installation is in conformance with its listing.

1705.16.2 Fire-resistant joint systems. Inspection of fire-resistant joint systems that are tested and listed in accordance with Sections 715.3 and 715.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

- A "Joint" is defined as a "linear opening in or between adjacent fire-resistance-rated assemblies that is designed to allow independent movement of the building in any plane caused by thermal, seismic, wind or any other loading." The joint creates an interruption of the fire-resistant integrity of the wall or floor system, requiring the use of an appropriate fire-resistant joint system. The code mandates general installation criteria for such systems and requires them to be tested in accordance with ASTM E 1966 or UL 2079. Much like the inspection of penetration firestop systems, the proper choice and installation of fire-resistant joint systems can be verified through a comprehensive special inspection process.

Although regulated under the provisions for fire-resistant joint systems, a second type of system is technically not a joint but rather an extension of protection afforded by a horizontal assembly. The void created at the intersection of an exterior curtain wall assembly and a fire-resistance-rated floor or floor/ceiling assembly must be filled in a manner that maintains the integrity of the horizontal assembly. The system utilized to fill the void must be in compliance with ASTM E 2307 and able to resist the passage of flame for a time period equal to that of the floor assembly. Special inspection is necessary to verify that the appropriate joint system is chosen and installed.

[F] 1705.17 Special inspection for smoke control. Smoke control systems shall be tested by a special inspector.

- Because smoke control systems are unique and complex life safety systems, this section requires that all smoke control systems be tested by special inspection.

[F] 1705.17.1 Testing scope. The test scope shall be as follows:

1. During erection of ductwork and prior to concealment for the purposes of leakage testing and recording of device location.
2. Prior to occupancy and after sufficient completion for the purposes of pressure difference testing, flow measurements and detection and control verification.

- Special inspections need to occur at two different stages during the installation of a smoke control system. The first round of special inspections occurs before concealment of the ductwork or fire protection elements. At this stage, the special inspector needs to verify that duct leakage is in accordance with Section 909.10.2. Additionally, the location of all fire protection devices needs to be verified and documented at this time. The second round of special inspections occurs just prior to occupancy in order to more closely replicate the conditions under which the system must operate. The inspections include the verification of pressure differences across smoke barriers as required in Sections 909.5.1 and 909.18.6, the verification of appropriate volumes of airflow as noted in the design, and finally the verification of the appropriate operation of the detection and control mechanisms as required in Sections 909.18.1 and 909.18.7 (see Section 909.18.8.3 for report requirements).

[F] 1705.17.2 Qualifications. Special inspection agencies for smoke control shall have expertise in fire protection engineering, mechanical engineering and certification as air balancers.

- This provision establishes a certain level of qualifications for the inspection of smoke control systems that would include the need for expertise in fire protection engineering, mechanical engineering and certification as air balancers.

SECTION 1706
DESIGN STRENGTHS OF MATERIALS

1706.1 Conformance to standards. The design strengths and permissible stresses of any structural material that are identified by a manufacturer’s designation as to manufacture and grade by mill tests, or the strength and stress grade is otherwise confirmed to the satisfaction of the building official, shall conform to the specifications and methods of design of accepted engineering practice or the approved rules in the absence of applicable standards.

- Structural materials must conform to applicable design standards, approved rules and accepted methods of engineering practice. Conformance to these provisions and to the manufacturer’s designations provides the building official with the information needed to verify that the materials will perform their intended function satisfactorily.

1706.2 New materials. For materials that are not specifically provided for in this code, the design strengths and permissible stresses shall be established by tests as provided for in Section 1707.

- Materials that are not explicitly covered by the code are allowed when subjected to the appropriate testing demonstrating adequate performance (see Section 1701.2).

SECTION 1707
ALTERNATIVE TEST PROCEDURE

1707.1 General. In the absence of approved rules or other approved standards, the building official shall make, or cause to be made, the necessary tests and investigations; or the building official shall accept duly authenticated reports from approved agencies in respect to the quality and manner of use of new materials or assemblies as provided for in Section 104.11. The cost of all tests and other investigations required under the provisions of this code shall be borne by the applicant.

- Test reports from approved agencies may be used as a basis for approval of materials that are not within the purview of any approved rules (i.e., "new materials" as mentioned in Section 1701.2). This section directly references Section 104.11. It is within the power of the building official to accept reports from an approved agency. In determining the approval, the building official should check that the agency is an independent third-party agency with no financial or fiduciary affiliations with the applicant or material supplier. The capability and competency of the agency must also be examined. It should be noted that this section assigns responsibility for the costs of testing to the applicant.

SECTION 1708
TEST SAFE LOAD

1708.1 Where required. Where proposed construction is not capable of being designed by approved engineering analysis, or where proposed construction design method does not comply with the applicable material design standard, the system of construction or the structural unit and the connections shall be subjected to the tests prescribed in Section 1710. The building official shall accept certified reports of such tests conducted by an approved testing agency, provided that such tests meet the requirements of this code and approved procedures.

- Testing to determine safe load is required when a structural component cannot be designed in accordance with approved engineering practices or where the construction design method does not fully comply with the respective material design standard listed in Chapter 35. If either of these situations exist, the structural components are required to be subjected to the prescriptive tests listed in Section 1710, which address loading and deflection criteria.

An example of a structural component that cannot be designed by approved engineering practice is a composite concrete and steel slab in which the shear connector is some type of new configuration called a "widget." The horizontal shear that can be developed is unknown; therefore, a complete analysis cannot be performed. This section also restates the building official’s option of accepting data from an approved testing agency, as previously stated in Section 1703.4.
SECTION 1709
IN-SITU LOAD TESTS

1709.1 General. Whenever there is a reasonable doubt as to the stability or load-bearing capacity of a completed building, structure or portion thereof for the expected loads, an engineering assessment shall be required. The engineering assessment shall involve either a structural analysis or an in-situ load test, or both. The structural analysis shall be based on actual material properties and other as-built conditions that affect stability or load-bearing capacity, and shall be conducted in accordance with the applicable design standard. If the structural assessment determines that the load-bearing capacity is less than that required by the code, load tests shall be conducted in accordance with Section 1709.2. If the building, structure or portion thereof is found to have inadequate stability or load-bearing capacity for the expected loads, modifications to ensure structural adequacy or the removal of the inadequate construction shall be required.

- The intent of this section is to utilize an engineering analysis to verify the adequacy of the structure, if possible. The load test requirement should only be done if an engineering analysis does not verify structural adequacy. Load tests are last resort options, and the building official should document his or her reasons for any load testing requirement.

An example of the executed structural analysis would be an analysis by a third-party engineering firm acceptable to both the building official and owner. The structural integrity may be questioned for items such as visible signs of excessive settlement or lateral deflection, such as cracks in concrete foundation walls or excessive vibration when the assembly is loaded.

A load test procedure must simulate the actual load conditions to which the structure is subjected during normal use (see Section 1709.3 for details).

1709.2 Test standards. Structural components and assemblies shall be tested in accordance with the appropriate referenced standards. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed by a registered design professional and approved. The test procedure shall simulate loads and conditions of application that the completed structure or portion thereof will be subjected to in normal use.

- When load test procedures for materials are given by the applicable referenced material standard, the test procedure outlined in that specific standard must be adhered to without variation. If a referenced standard lacks a load test procedure, or a material or assembly does not have a specific referenced standard, then such a test must be developed by a registered design professional and approved by the building official. The test procedure must be representative of, and simulate the actual loading conditions that, the completed structure or portion thereof will be subjected to during normal use.

1709.3 In-situ load tests. In-situ load tests shall be conducted in accordance with Section 1709.3.1 or 1709.3.2 and shall be supervised by a registered design professional. The test shall simulate the applicable loading conditions specified in Chapter 16 as necessary to address the concerns regarding structural stability of the building, structure or portion thereof.

- The criteria for in-situ load tests are set forth for two categories: procedures specified, which are regulated by Section 1709.3.1, and procedures not specified, which are regulated by Section 1709.3.2. This section further requires that the test be performed under the supervision of a registered design professional and that it simulates the actual loads and conditions of the completed structure or portion thereof.

1709.3.1 Load test procedure specified. Where a referenced standard contains an applicable load test procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of specific load factors or acceptance criteria, the load factors and acceptance criteria in Section 1709.3.2 shall apply.

- The load test must be in accordance with the applicable referenced standard. Section 1709.3.2 must only be utilized in the absence of either a specific standard or specific load factors and acceptance criteria from applicable referenced standards.

1709.3.2 Load test procedure not specified. In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such existing structure shall be subjected to a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components that are not a part of the seismic load-resisting system, the test load shall be equal to two times the unfactored design loads. The test load shall be left in place for a period of 24 hours. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

- If the applicable standards do not specify load factor or testing criteria acceptance methods, then the testing criteria listed in this section must be followed. Note that the design load includes design live load and all dead loads that are not yet in place, such as the dead load from tenant walls in a speculative office building.

SECTION 1710
PRECONSTRUCTION LOAD TESTS

1710.1 General. In evaluating the physical properties of materials and methods of construction that are not capable of being designed by approved engineering analysis or do not
comply with the applicable referenced standards, the structural adequacy shall be predetermined based on the load test criteria established in this section.

- This section establishes requirements for load testing structural assemblies that are either incapable of being designed or those that, for one reason or another, do not comply with the applicable material design standards. This section does not govern the load testing of existing buildings, which is governed by Section 1709.

The different categories of preconstruction load tests are addressed herein. Specified load test procedures are regulated by Section 1710.2. Load test procedures that are not specified are regulated by Section 1710.3. Wall and partition assemblies are regulated by Section 1710.4. Exterior window and door assemblies are regulated by Section 1710.5. Skylights and sloped glazed are regulated by Section 1710.6 and test specimens are regulated by Section 1710.7.

**1710.2 Load test procedures specified.** Where specific load test procedures, load factors and acceptance criteria are included in the applicable referenced standards, such test procedures, load factors and acceptance criteria shall apply. In the absence of specific test procedures, load factors or acceptance criteria, the corresponding provisions in Section 1710.3 shall apply.

- This section has priority over Section 1710.3, provided that load factors and acceptance criteria are established in the applicable design standards.

**1710.3 Load test procedures not specified.** Where load test procedures are not specified in the applicable referenced standards, the load-bearing and deformation capacity of structural components and assemblies shall be determined on the basis of a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components and assemblies that are not a part of the seismic force-resisting system, the test shall be as specified in Section 1710.3.1. Load tests shall simulate the applicable loading conditions specified in Chapter 16.

- In the absence of load factors and acceptance criteria in the applicable design standards and in accordance with Section 1710.2, this section is to be used by the building official to determine if conformance to the applicable code requirements has been achieved. Additionally, this section provides the design professional and building official with specific loading and pass/fail criteria.

**1710.3.1 Test procedure.** The test assembly shall be subjected to an increasing superimposed load equal to not less than two times the superimposed design load. The test load shall be left in place for a period of 24 hours. The tested assembly shall be considered to have successfully met the test requirements if the assembly recovers not less than 75 percent of the maximum deflection within 24 hours after the removal of the test load. The test assembly shall then be reloaded and subjected to an increasing superimposed load until either structural failure occurs or the superimposed load is equal to two and one-half times the load at which the deflection limitations specified in Section 1710.3.2 were reached, or the load is equal to two and one-half times the superimposed design load. In the case of structural components and assemblies for which deflection limitations are not specified in Section 1710.3.2, the test specimen shall be subjected to an increasing superimposed load until structural failure occurs or the load is equal to two and one-half times the desired superimposed design load. The allowable superimposed design load shall be taken as the lesser of:

1. The load at the deflection limitation given in Section 1710.3.2.
2. The failure load divided by 2.5.
3. The maximum load applied divided by 2.5.

- Load test criteria relating to superimposed design loads are established herein. These requirements are a compilation of commonly accepted engineering practices to adequately test against structural failure.

In the case of structural components and assemblies for which maximum deflection limitations are not addressed in Section 1710.3.2, the test assemblies must be subjected to increasing superimposed loads until failure occurs or the load is equal to two and one-half times the superimposed design load, whichever occurs first.

**1710.3.2 Deflection.** The deflection of structural members under the design load shall not exceed the limitations in Section 1604.3.

- Acceptance criteria for deflection of structural systems when subjected to the allowable design load are used to demonstrate adequate structural performance and are addressed in Section 1604.3.

**1710.4 Wall and partition assemblies.** Load-bearing wall and partition assemblies shall sustain the test load both with and without window framing. The test load shall include all design load components. Wall and partition assemblies shall be tested both with and without door and window framing.

- Load-bearing wall and partition assemblies must sustain loads with and without window framing. It is not appropriate to assume that a wall will sustain loads better if window framing is involved in the test. Each individual design must be evaluated separately based on the construction of that assembly. All design loads, such as vertical and lateral forces, must be included in the test.

**1710.5 Exterior window and door assemblies.** The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section 1710.5.1 or 1710.5.2.

**Exception:** Structural wind load design pressures for window units smaller than the size tested in accordance with Section 1710.5.1 or 1710.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design
pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

This section allows two methods of load testing for exterior window and door assemblies. The first method, provided for in Section 1710.5.1, allows products to be tested and labeled as conforming to AAMA/WMDA/CSA101/LS.2/A440. The second method allows products to be tested in accordance with ASTM E 330 and the glazing must comply with Section 2403. The exception allows window units smaller than the size tested to have higher design pressures, provided the higher pressures are determined by accepted engineering analysis, all components of the smaller unit are the same as the tested unit and an additional test of the smaller unit having the highest calculated design pressure is performed in accordance with Section 1710.5.1 or 1710.5.2.

1710.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WMDA/CSA101/LS.2/A440. The label shall state the name of the manufacturer, the approved labeling agency, and the product designation as specified in AAMA/WMDA/CSA101/LS.2/A440. Exterior side-hinged doors shall be tested and labeled as conforming to AAMA/WMDA/CSA101/LS.2/A440 or comply with Section 1710.5.2. Products tested and labeled as conforming to AAMA/WMDA/CSA 101/LS.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.

This section requires exterior windows and doors complying with AAMA/WMDA/CSA 101/LS.2/A440 to be labeled as such. Products so tested and labeled must not be required to meet the provisions of Sections 2403.2 and 2403.3. By requiring the product to be labeled, the building official does not have to interpret test results and determine load-carrying capacities, or accept the manufacturer's interpretation of tests.

1710.5.2 Exterior windows and door assemblies not provided for in Section 1710.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of garage doors and rolling doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for 10 seconds at a load equal to 1 1/2 times the design pressure.

This section allows an alternative to the provisions of Section 1710.5.1. This procedure is to be used to verify the integrity of door and window assemblies as a whole, and its results do not supersede, but rather complement, the requirements of Chapter 24. Glass thickness must be determined in accordance with the provisions of Chapter 24. The assemblies must comply with Section 2403 and the design pressure for testing is determined from Chapter 16. Each assembly is required to be tested for 10 seconds at a load equal to one and one-half times the design pressure. When testing a product that has a variety of sizes, the most critical size can usually be tested and the results used to qualify other similar products within its family. In general, the larger size and most heavily loaded of each particular design, type, construction or configuration should be tested. The code does not specify how many specimens are to be tested. ASTM E 330 states that if only one sample is tested, it should be selected by the specifying authority. ANSI/DASMA 108 addresses the pressure testing of garage doors and includes garage door acceptance criteria, which is not provided by ASTM E 330.

1710.6 Skylights and sloped glazing. Unit skylights and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2405. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

A tubular daylighting device (TDD) is typically field-assembled from a manufactured kit, unlike a unit sky-light which is typically shipped as a factory-assembled unit. The dome of a TDD is not necessarily constructed out of a single panel of glazing material. Thus, a separate definition of a TDD was added to Chapter 2 based on the definition in AAMA/WMDA A440. The section refers to Section 2405 for sloped unit skylight and TDD requirements.

1710.7 Test specimens. Test specimens and construction shall be representative of the materials, workmanship and details normally used in practice. The properties of the materials used to construct the test assembly shall be determined on the basis of tests on samples taken from the load assembly or on representative samples of the materials used to construct the load test assembly. Required tests shall be conducted or witnessed by an approved agency.

The test specimen must resemble and simulate as much as possible, the design being tested using materials and workmanship that could be expected in the actual construction or fabrication. The test itself must be witnessed or conducted by an agency acceptable to and approved by the building official.

SECTION 1711
MATERIAL AND TEST STANDARDS

1711.1 Joist hangers. Testing of joist hangers shall be in accordance with Sections 1711.1.1 through 1711.1.3, as applicable.

This section prescribes the test standard and criteria to be used for joist hangers and connectors. This criterion is meant to be used in establishing the load capacity of joist hangers and connectors used in wood construction for which there is no calculated procedure recognized by the code.

The referenced test standard, ASTM D 1761, requires the joist length to be twice the joist depth plus 10 inches (254 mm), but this can result in longer joists that fail before the hanger. In establishing a
hanger's allowable load, the objective is to have the hanger fail in the test, rather than the joist. The exception sets a maximum joist length in order to facilitate the appropriate failure mode.

1711.1.1 General. The vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with ASTM D 1761 using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AF&PA NDS for the joist and headers.

Exception: The joist length shall not be required to exceed 24 inches (610 mm).

The specified ASTM standard test method provides a procedure for evaluating the vertical load-carrying capacity, torsional moment capacity and deflection characteristics of joist hangers and similar devices used to connect wood joists to headers of wood or other materials. The lumber used for the test specimen must have a specific gravity equal to or greater than 0.49, but not greater than 0.55.

1711.1.2 Vertical load capacity for joist hangers. The vertical load-bearing capacity for the joist hanger shall be determined by testing a minimum of three joist hanger assemblies as specified in ASTM D 1761. If the ultimate vertical load for any one of the tests varies more than 20 percent from the average ultimate vertical load, at least three additional tests shall be conducted. The allowable vertical load-bearing of the joist hanger shall be the lowest value determined from the following:

1. The lowest ultimate vertical load for a single hanger from any test divided by three (where three tests are conducted and each ultimate vertical load does not vary more than 20 percent from the average ultimate vertical load).

2. The average ultimate vertical load for a single hanger from all tests divided by three (where six or more tests are conducted).

3. The average from all tests of the vertical loads that produce a vertical movement of the joist with respect to the header of \(1/8\) inch (3.2 mm).

4. The sum of the allowable design loads for nails or other fasteners utilized to secure the joist hanger to the wood members and allowable bearing loads that contribute to the capacity of the hanger.

5. The allowable design load for the wood members forming the connection.

The method prescribed establishes the allowable load for normal duration, as defined by the American Forest & Paper Association (AF&PA) National Design Specification (NDS) for Wood Construction. Additionally, allowable stresses cannot exceed those allowed by the code. For example, published allowable loads cannot contain nail loads higher than those allowed by AF&PA NDS, nor can tension in steel strapping exceed that allowed by the steel design standards noted in Chapter 22.

For loads of other than normal duration, the stresses or loads may be increased or must be decreased as noted by the appropriate design standard, but in no case can the load exceed that which will produce \(1/6\)-inch (3.2 mm) movement of the joist.

EXAMPLE:

Given:

A manufacturer's test results for a particular joist hanger are as follows:

**Test 1** Ultimate load = 1,000 pounds with the \(1/6\)-inch deflection occurring at 400 pounds.

**Test 2** Ultimate load = 1,100 pounds with the \(1/6\)-inch deflection occurring at 350 pounds.

**Test 3** Ultimate load = 900 pounds with the \(1/6\)-inch deflection occurring 375 pounds.

For SI: 1 inch = 25.4 mm, 1 pound = 0.454 kg.

The manufacturer submitted structural calculations indicating that the allowable design load of the nails is 250 pounds (114 kg), and the allowable shear load in the wood joists framing to the hangers is 280 pounds (127 kg). Joist hanger geometry does not allow for any meaningful calculation of stresses in the steel sections of the hanger.

**Find:** The allowable load for the joist hanger.

**Solution:**

Average ultimate load = \((1,000 + 1,100 + 900) ÷ 3 = 1,000 \text{ pounds (454 kg)}\)

Since 20 percent of 1,000 is 200, the test scatter is within the allowable range of plus and minus 20 percent of the average ultimate load; therefore, three tests are sufficient to establish allowable load.

Thus, the allowable load is 250 pounds (114 kg) based on the lesser of:

- Lowest ultimate load + 3.0 = 300 pounds (137 kg)
- \(1/6\)-inch deflection in any test = 350 pounds (159 kg)

Allowable nail load = 250 pounds (114 kg)

Allowable joist shear = 280 pounds (127 kg)

In this case, the allowable vertical load for a normal duration is limited by the calculated allowable design load of the fastener [250 pounds (113 kg)] in accordance with Item 4 of Section 1711.1.2. This value is permitted to be modified by a duration of loading factor; however, the modified value cannot exceed the lowest test value as determined in accordance with Items 1, 2 and 3 of Section 1711.1.2.

1711.1.2.1 Design value modifications for joist hangers. Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 1711.1.2 shall be permitted to be modified by the appropriate load duration factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by Item 1, 2 or 3 in Section 1711.1.2. Allowable
design values determined by Item 1, 2 or 3 in Section 1711.1.2 shall not be modified by load duration factors.

- The calculated allowable design values, as determined by Item 4 or 5 of Section 1711.1.2 and modified by duration of loading factors, must not exceed the lowest test value as determined by Item 1, 2 or 3 of Section 1711.1.2.

1711.1.3 Torsional moment capacity for joist hangers. The torsional moment capacity for the joist hanger shall be determined by testing at least three joist hanger assemblies as specified in ASTM D 1761. The allowable torsional moment of the joist hanger shall be the average torsional moment at which the lateral movement of the top or bottom of the joist with respect to the original position of the joist is \( \frac{1}{4} \) inch (3.2 mm).

- The allowable torsional moment capacity for a joist hanger is determined by testing in accordance with ASTM D 1761 with the limitation that rotational deflection of the top or bottom of the joist with respect to the header must not exceed 0.125 inch (3.2 mm).

1711.2 Concrete and clay roof tiles. Testing of concrete and clay roof tiles shall be in accordance with Sections 1711.2.1 and 1711.2.2, as applicable.

- This section prescribes the test standards and criteria to be used to determine the overturning resistance and wind characteristics of concrete and clay roof tiles.

1711.2.1 Overturning resistance. Concrete and clay roof tiles shall be tested to determine their resistance to overturning due to wind in accordance with SBCCI SSTD 11 and Chapter 15.

- Section 1711.2.1 requires concrete and clay tiles to be tested to determine their overturning resistance in accordance with SBCCI SSTD 11. SBCCI SSTD 11 prescribes methods for determining the allowable overturning moment for mechanically fastened, adhesive-set and mortar-set tiles. A test procedure is also prescribed for determining the allowable uplift loads on hip/ridge tiles.

1711.2.2 Wind tunnel testing. Where concrete and clay roof tiles do not satisfy the limitations in Chapter 16 for rigid tile, a wind tunnel test shall be used to determine the wind characteristics of the concrete or clay tile roof covering in accordance with SBCCI SSTD 11 and Chapter 15.

- The wind tunnel test procedures in SBCCI SSTD 11 must be used if the roof tiles do not meet the limitations of Chapter 16 for rigid tile.

Bibliography

The following resource materials are referenced in this chapter or are relevant to the subject matter addressed in this chapter.

- ACI 318-11, Building Code Requirements for Structural Concrete. Farmington Hills, MI: American Concrete Institute, 2011.


Chapter 18: Soils and Foundations

General Comments

Chapter 18 contains provisions regulating the design and construction of foundations for buildings and other structures and involves geotechnical and structural considerations in the selection and installation of adequate supports for the loads transferred from the structure above.

Section 1801 gives the general scope or purpose of the provisions in Chapter 18.

Section 1802 lists definitions that pertain to foundations.

Section 1803 provides the requirements for foundation and geotechnical investigations that are to be conducted at the site prior to design and construction.

Section 1804 includes excavation, grading and backfill provisions.

Section 1805 provides specifications for the damp-proofing and waterproofing of floor slabs and below-grade walls.

Section 1806 establishes the allowable load-bearing values for soils where site soil tests do not verify that higher soil values are appropriate.

Section 1807 provides requirements for foundation walls, retaining walls and embedded posts.

Section 1808 gives general requirements for foundations.

Section 1809 provides the specifications for shallow foundations.

Section 1810 includes the provisions for deep foundations.

The proper design and construction of a foundation system is critical to the satisfactory performance of the entire building structure that it supports.

Foundation problems are not uncommon and vary greatly. They may be of a simple or complex nature and may be manageable or without practical remedy. The uncertainties of foundation construction make it extremely difficult to address every potential problem area within the text of the code. The provisions of the code are meant to set forth and regulate the minimum standards and conventional practices needed for the design and construction of foundation systems so as to provide adequate safety to life and property. Due care must be exercised in the planning and design of foundation systems based on obtaining sufficient soils information, the use of accepted engineering procedures, experience and good technical judgement.

Essentially, there are two parts to the foundation system: the substructure and the soil. The substructure consists of structural components that serve as the medium through which the building loads are transmitted to the supporting earth (soil or rock). The substructure components may consist of shallow foundations, such as basement walls, grade walls, beams, isolated spread footings or combinations of these components. Shallow foundations may also involve the use of mat or raft foundations. As may be required, the substructure can consist of deep foundations involving the use of piles; drilled shafts or piers; caissons or other similar deep foundations. The second part of the foundation system involves the use of soil (including rock) as a structural material to carry the load of the building or any other load transmitted through the substructure.

The substructure and the soil are interdependent elements of the foundation system and must be understood and dealt with as a composite engineering consideration. Indeed, the selection of the kind of substructure to be used, whether it employs any of the different types of shallow foundations commonly used or adopts the use of a deep foundation, is a direct function of the nature of the soil encountered at the project site. Chapter 18 broadly outlines the conventional systems of foundation construction. Although it does not specifically include special or patented systems, it does not preclude their use because most such construction will fall into the general categories of foundation systems prescribed in the provisions and, thus, meet the intent of the code.

In determining the load-bearing capacity and other values of the soil mass, the code provisions address such considerations in terms of "undisturbed" soil. Special provisions are included where prepared fill is to be utilized for foundation support.

Purpose

The provisions of this chapter set forth the minimum requirements for the design and construction of foundation systems for buildings and other structures.
SECTION 1801
GENERAL

1801.1 Scope. The provisions of this chapter shall apply to building and foundation systems.

- The provisions contained in this chapter for foundation design and construction apply to all structures.

1801.2 Design basis. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1803.3. The quality and design of materials used structurally in excavations and foundations shall comply with the requirements specified in Chapters 16, 19, 21, 22 and 23 of this code. Excavations and fills shall also comply with Chapter 33.

- Design requirements in Chapter 18 are generally based on an allowable stress design (ASD) approach. Allowable stresses and service loads should not be used with the load combinations for strength design, and vice versa. This section clarifies the applicable load combinations from Chapter 16 that are to be used for the design of foundations.

SECTION 1802
DEFINITIONS

1802.1 Definitions. The following words and terms are defined in Chapter 2:

DEEP FOUNDATION.

DRILLED SHAFT.

Socketed drilled shaft.

HELICAL PILE.

MICROPILE.

SHALLOW FOUNDATION.

- Definitions are intended to facilitate the understanding of code provisions and to minimize potential confusion. To that end, this section lists the definitions of terms associated with foundations that can be found in Chapter 2. The use and application of all defined terms, as well as undefined terms, are set forth in Section 201.

SECTION 1803
GEO TECHNICAL INVESTIGATIONS

1803.1 General. Geotechnical investigations shall be conducted in accordance with Section 1803.2 and reported in accordance with Section 1803.6. Where required by the building official or where geotechnical investigations involve in-situ testing, laboratory testing or engineering calculations, such investigations shall be conducted by a registered design professional.

- This section addresses the conditions that mandate a geotechnical investigation, as well as the information that must be included in the report. The investigation of soils is to be done by a registered design professional in recognition that the testing and calculations necessitate individuals with significant experience in soil and foundation analysis. The field of soil mechanics and foundation engineering is diverse and complicated, and since it is not an exact science, its application requires specialized knowledge and judgment based on experience. Where subsurface conditions are found or suspected to be of a critical nature, the building official is encouraged to seek the professional advice of experienced foundation engineers.

1803.2 Investigations required. Geotechnical investigations shall be conducted in accordance with Sections 1803.3 through 1803.5.

Exception: The building official shall be permitted to waive the requirement for a geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 1803.5.1 through 1803.5.6 and Sections 1803.5.10 and 1803.5.11.

- Soils investigations to determine subsurface conditions should be made prior to the design and construction of new buildings and other structures. Such investigations should also be conducted when additions to existing facilities are considered and are of such a scope that would significantly increase or change the distribution of foundation loads.

There are two main objectives for conducting a soils investigation. The first is of a confirmatory nature. Its purpose is to obtain information already known from adjacent structures, such as soil-boring records, field test results, laboratory test data and analyses and any other knowledge useful in the design of the foundation system. The second objective is of an exploratory nature. It is warranted where soils information does not exist or is insufficient or unsatisfactory for use in the design of the foundation system.

Regardless of the objective of the soils investigation, the information generally required includes one or more (or all) of the following items for determining subsurface conditions:

1. The depth, thickness and composition of each soil stratum;

2. For rock, the characteristics of the rock stratum (or strata), including the thickness of the rock to a reasonable depth;

3. The depth of ground water below the site surface; and

4. The engineering properties of the soil and rock strata that are pertinent for the proper design and performance of the foundation system.

For shallow foundations, the soils investigation should yield sufficient information to establish the character and load-bearing capacity of the soil (or rock) at depths that will receive the foundations. Foundation problems are not uncommon and may vary greatly, ranging from very simple and manage-
able problems to very complex situations that may be either manageable or without practical remedy.

As indicated in the exception, where geotechnical data from adjacent areas are well known, the building official can accept the use of local engineering practices for the design of foundations.

1803.3 Basis of investigation. Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

❖ When soils are required to be classified, the classification must be based on observations and tests, such as borings or test pits. In addition to the situations specified that require a soils investigation and classification, the evaluation of slope stability, soil strength, position and adequacy of load-bearing soils, moisture effects, compressibility and liquefaction are required to be performed when deemed necessary by the building official or registered design professional.

1803.3.1 Scope of investigation. The scope of the geotechnical investigation including the number and types of borings or soundings, the equipment used to drill or sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional.

❖ Whenever the load capacity of a soil is in doubt and a field investigation is necessary, exploratory borings are to be made to determine the load-bearing value of the soil. The investigation is to be performed by a registered design professional, which in most cases would be a geotechnical engineer. Exploratory borings and their associated tests should be conducted in each area of relatively dissimilar subsurface conditions.

1803.4 Qualified representative. The investigation procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on site during all boring or sampling operations.

❖ A qualified representative is required on site to verify that the information obtained from the investigation will be adequate, valid and acceptable to the building official.

1803.5 Investigated conditions. Geotechnical investigations shall be conducted as indicated in Sections 1803.5.1 through 1803.5.12.

❖ Sections 1803.5.1 through 1803.5.12 state conditions that necessitate a subsurface investigation.

1803.5.1 Classification. Soil materials shall be classified in accordance with ASTM D 2487.

❖ Where required, soils are to be classified in accordance with ASTM D 2487. This standard provides a system for classifying soils for engineering purposes based on laboratory determination of particle size characteristics, liquid limit and plasticity index. The

classification system identifies three major soil divisions—course-grained, fine-grained and highly organic—which are separated further into 15 basic soil groups. ASTM D 2487 is the ASTM International version of the Unified Soil Classification System.

1803.5.2 Questionable soil. Where the classification, strength or compressibility of the soil is in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall be permitted to require that a geotechnical investigation be conducted.

❖ Whenever relevant soil characteristics are in doubt, or where a design is based upon load-bearing values that are greater than those specified in the code, the building official may require investigation and testing of the soil.

One such method of investigation includes construction test pits for field load-bearing tests. Test pits are usually required to be at least 4 square feet (0.37 m²) in area and be excavated down to the elevation of the proposed bearing surface. The typical apparatus for making such tests involves placing the test loads on a platform supported on a post through which the applied loads are transferred to a bearing plate of a specified size and, in turn, to the soil below. A typical setup for field load tests is shown in Figure 1803.5.2.

It is important that the load (weights on platform) is applied such that all of it will be transmitted to the soil as a static load without impact, fluctuation or eccentricity. The load should be applied incrementally, and continuous records of all settlements should be kept. Measurements are usually made by settlement

![Figure 1803.5.2 TYPICAL SETUP FOR CONDUCTING STATIC LOAD TESTS](image)
SOILS AND FOUNDATIONS

recording devices, such as dial gauges, capable of measuring the settlement of the test bearing plate to an accuracy of at least 0.01 inch (0.25 mm).

The test is continued until either the maximum test load is reached or the ratio of load increment to settlement increment reaches a minimum, steady magnitude sustained for a period of 48 hours. After the load is released, the elastic rebound of the soil is also measured for a period of time.

Load test results are normally presented in a load settlement diagram in which the applied test load measured in tons per square foot is plotted in relation to the settlement readings recorded in fractions of an inch. The bearing capacity of the soil can be computed from the test results.

There are some drawbacks to the use of field load tests for determining soil-bearing capacity. Test results can be misleading if the soil under the footing is not uniform for the full depth of load influence, which is equal to about twice the width of the footing. Also, since a load test is conducted for a short duration, settlements that occur due to the consolidation of the soil over a very long time cannot be predicted. Since this type of field load-bearing testing is relatively expensive, it is not widely used.

Other methods for determining the safe bearing capacity of soils may be more appropriate. Standard laboratory tests can usually produce sufficient proof of satisfactory bearing capacity and settlement information. However, there are conditions when standard laboratory tests may not produce reliable results, such as when clay materials contain a pattern of cracks or when stiff clays may have suffered differential movement or expansion. As stated in Section 1803.3.1, a registered design professional must establish an appropriate testing program.

1803.5.3 Expansive soil. In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist.

Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

Expansive soils, often referred to as "swelling soils," contain montmorillonite minerals and have the characteristics of absorbing water and swelling, or shrinking and cracking when drying. Significant volume changes can cause serious damage to buildings and other structures as well as to pavements and sidewalks. Swelling soils are found throughout the nation, but are more prevalent in regions with dry or moderately arid climates.

There is a general relationship between the plasticity index (PI) of a soil as determined by the ASTM D 4318 standard test method and the potential for expansion, as shown in Figure 1803.5.3.

This section defines "Expansive soil" as any plastic material with a PI of 15 or greater (Item 1); with more than 10 percent of the soil particles passing a No. 200 sieve (Item 2); less than 5 micrometers in size (Item 3) and having an expansion index (EI) greater than 20 (Item 4). Alternatively, the EI in accordance with ASTM D 4829 can be used exclusively. The EI value is a measure of the swelling potential of the soil. A soil with an EI value of 20 or less has a very low potential for expansion.

The amount and depth of potential swelling that can occur in a clay material are, to some extent, functions of the cyclical moisture content in the soil. In dryer climates where the moisture content in the soil near the ground surface is low because of evaporation, there is a greater potential for extensive swelling than the same soil in wetter climates where the variations of moisture content are not as severe. Volume changes in highly expansive soils range between 7 and 10 percent, but experience has shown that occasionally, under abnormal conditions, they can reach as high as 25 percent.

### Table: Swelling Potential Plasticity Index

<table>
<thead>
<tr>
<th>Plasticity Index</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0-15</td>
</tr>
<tr>
<td>Medium</td>
<td>10-35</td>
</tr>
<tr>
<td>High</td>
<td>20-55</td>
</tr>
<tr>
<td>Very high</td>
<td>35 and above</td>
</tr>
</tbody>
</table>

Figure 1803.5.3

**Swelling Potential of Soils and Plasticity Index**


1803.5.4 Ground-water table. A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

**Exception:** A subsurface soil investigation to determine the location of the ground-water table shall not be required.
where waterproofing is provided in accordance with Section 1805.

- There are several reasons for conducting a subsurface investigation to determine the level of ground water at a construction site. If the ground-water table is above subsurface slabs (i.e., basement floors), then walls and floors need to be designed to resist hydrostatic pressures. Foundation walls and basement slabs may need to be damproofed or waterproofed, depending on the location of the ground-water table. A subsurface investigation will also determine the type of drainage system needed as a permanent installation, whether there will be any major water problems that could affect the excavation operations and construction of the foundation system, and if it is necessary to provide a temporary drainage system of a type and size that will control ground-water seepage.

Ground-water levels can vary significantly over a year's time, as well as from year to year. While it would be ideal to make ground-water table observations that encompass a full annual cycle, the reality is that such an undertaking would not normally facilitate a design/construction program and would be impractical. Ground-water observations must be made in shorter time intervals; however, this situation poses some real problems. For example, measurements of ground-water levels in bore holes taken 24 hours after completion of the soil borings can provide an acceptable indication of the water level in permeable soils, such as sand, gravel or sand/gravel mixtures. Fine-grained soils of low permeability, such as clays, require the use of observation tubes (piezometers) and, depending on the specific properties of the soil, a time period of 10 weeks or longer to obtain acceptable readings.

Water levels established by either of the two methods described above are sufficient indication of the water conditions at the time of measurement, but do not necessarily represent the highest possible ground-water levels that can occur. For design purposes, the water levels established by field observations may need to be adjusted with the climatological and hydrological records of the region in order to establish the high and low points.

As indicated in the exception, a subsurface investigation is not required where floors, walls, joints and penetrations are waterproofed as required in Section 1805.3.

1803.5.5 Deep foundations. Where deep foundations will be used, a geotechnical investigation shall be conducted and shall include all of the following, unless sufficient data upon which to base the design and installation is otherwise available:

1. Recommended deep foundation types and installed capacities.
2. Recommended center-to-center spacing of deep foundation elements.
3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Load test requirements.
7. Suitability of deep foundation materials for the intended environment.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

- A foundation investigation is required when deep foundations are proposed. Such investigations are needed to define as accurately as possible the subsurface conditions of soil and rock materials, establish the soil and rock profiles across the construction site and locate the ground-water table. Sometimes, it may also be necessary to determine specific soil properties, such as shear strength, relative density, compressibility and other such technical data required for analyzing subsurface conditions. Foundation investigations may also be used to render such valuable data as information on existing construction at the site or on neighboring properties (including boring and test records), the type and condition of the existing structures, their age, the type of foundations used and performance over the years. Other helpful information includes knowledge of existing deleterious substances in the soils that could affect the durability (as well as the performance) of the piles, data on geologic conditions at the site (including such information as the existence of mines, earth cavities, underground streams or other adverse water conditions), as well as a history of any seismic activity.

The types of information described above are usually obtained by means of soil and rock borings; laboratory and field tests and engineering analyses. Such information is used for determining design loads; types and lengths of piles; driving criteria and selection of equipment and probable durability of pile materials in relation to subsurface conditions.

1803.5.6 Rock strata. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

- Rock may be found at levels near or at the earth's surface, upon which shallow foundations can be supported or will range downward to very low levels, upon which piles and other types of deep foundations can bear.

Most intact rock will have compressive strengths that far exceed the requirements for foundation support. It is most common, however, to find cracks,
SOILS AND FOUNDATIONS

joints and other defects in rock formations that will increase the compressibility of the material. Depending on the nature and extent of the defects, settlement may become the governing factor in determining allowable load-bearing capacity rather than rock strength.

Where the condition of the rock is in doubt, borings must be made at least 10 feet (305 mm) into the rock stratum below the bottom of the footings to verify the soundness of the material and to determine its load-bearing capacity.

1803.5.7 Excavation near foundations. Where excavation will remove lateral support from any foundation, an investigation shall be conducted to assess the potential consequences and address mitigation measures.

- Section 1804.1 addresses soil stability when excavations are made adjacent to existing foundations. This provision makes it clear that a geotechnical investigation is necessary in these situations.

1803.5.8 Compacted fill material. Where shallow foundations will bear on compacted fill material more than 12 inches (305 mm) in depth, a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test methods to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

- The information on the prepared fill in the soils report is necessary to permit a reasonable prediction as to the load-bearing capacity of the fill material. Where prepared fill is to be utilized for foundation support, the geotechnical report is to contain detailed information for approval of the fill. The information on the prepared fill in the soils report is necessary to permit a reasonable prediction as to the load-bearing capacity of the fill material. Additionally, when prepared fill is to be utilized where special inspections are required, the fill operation itself must be performed under the scrutiny of a special inspector. For example, if one (or more) of the exceptions in Section 1704.1 applies, special inspection of the prepared fill operation is not required.

1803.5.9 Controlled low-strength material (CLSM). Where shallow foundations will bear on controlled low-strength material (CLSM), a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

- As an alternative to compacted fill, the code permits the use of controlled low-strength material (CLSM) for the support of footings. CLSM must be placed in accordance with an approved report that includes requirements for the material strength and field verification.

In Chapter 2, "CLSM" is defined as "self-compacting cementitious materials." This class of material is commonly referred to by a variety of other names, which include flowable fill, controlled density fill, unshrinkable fill and soil-cement slurry. Guidance on the use of these materials can be found in ACI 229R. Additional documents that may be useful references for sampling and testing these materials include the following ASTM International standards:

ASTM D 6023, Standard Test Method for Unit Weight, Yield, Cement Content, and Air Content (Gravimetric) of Controlled Low-strength Material (CLSM).
ASTM D 6024, Standard Test Method for Ball Drop on Controlled Low-strength Material (CLSM) to Determine Suitability for Load Application.

1803.5.10 Alternate setback and clearance. Where setbacks or clearances other than those required in Section 1808.7 are desired, the building official shall be permitted to require a geotechnical investigation by a registered design professional to demonstrate that the intent of Section 1808.7 would be satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

- Section 1808.7 regulates the placement of foundations adjacent to slopes that are greater than one unit vertical to three units horizontal (33.3-percent slope). This section provides the building official the authority to approve alternative setbacks and clearances to
those required in Section 1808.7. The building official has the authority to require an investigation by a registered design professional to show that the intent of the code has been met. This item also specifies the parameters that must be considered by the registered design professional in the investigation.

1803.5.11 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F, a geotechnical investigation shall be conducted, and shall include an evaluation of all of the following potential geologic and seismic hazards:

1. Slope instability.
2. Liquefaction.
3. Total and differential settlement.
4. Surface displacement due to faulting or seismically induced lateral spreading or lateral flow.

The potential for liquefaction, surface rupture or slope instability at a building site is greater in areas of moderate and high seismicity than in areas of low seismicity. Also, the consequences of damage resulting from such hazards are more severe for buildings in higher risk categories (such as essential facilities). Thus, this section requires an investigation report for building sites that are assigned to Seismic Design Category C or higher, that includes an evaluation of the specific earthquake hazards that are listed. The purpose of this section is to reduce the hazard of large ground movement and the damaging effects on the structure.

Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. For example, many structures in Niigata, Japan suffered major damage as a consequence of liquefaction during its 1964 earthquake. Loss of bearing strength, differential settlement and differential horizontal displacement due to lateral spread were the direct causes of damage. Many structures have been similarly damaged by differential ground displacement during U.S. earthquakes, such as the San Fernando Valley Juvenile Hall during the 1971 San Fernando, California earthquake and the Marine Sciences Laboratory at Moss Landing, California during the 1989 Loma Prieta event.

For more information regarding evaluation of slope instability, liquefaction and surface rupture due to faulting or lateral spreading, see Section 7.4 of the National Earthquake Hazards Reduction Program (NEHRP) Provisions commentary (FEMA 450).

1803.5.12 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, the geotechnical investigation required by Section 1803.5.11 shall also include all of the following as applicable:

1. The determination of dynamic seismic lateral earth pressures on foundation walls and retaining walls supporting more than 6 feet (1.83 m) of backfill height due to design earthquake ground motions.

2. The potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the maximum considered earthquake ground motions. Peak ground acceleration shall be determined based on:
   2.1 A site-specific study in accordance with Section 21.5 of ASCE 7; or
   2.2 In accordance with Section 11.8.3 of ASCE 7.

3. An assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to:
   3.1. Estimation of total and differential settlement;
   3.2. Lateral soil movement;
   3.3. Lateral soil loads on foundations;
   3.4. Reduction in foundation soil-bearing capacity and lateral soil reaction;
   3.5. Soil downdrag and reduction in axial and lateral soil reaction for pile foundations;
   3.6. Increases in soil lateral pressures on retaining walls; and
   3.7. Flotation of buried structures.

4. Discussion of mitigation measures such as, but not limited to:
   4.1. Selection of appropriate foundation type and depths;
   4.2. Selection of appropriate structural systems to accommodate anticipated displacements and forces;
   4.3. Ground stabilization; or
   4.4. Any combination of these measures and how they shall be considered in the design of the structure.

This section includes additional requirements for the soil investigation report for sites with structures assigned to Seismic Design Categories D and higher. The investigation must determine lateral earth pressures on basement and retaining walls due to earthquake motions. Earthquake motions create increased lateral soil pressure on walls below the ground surface, especially in soft soils in areas of high seismicity. This requirement makes certain that the dynamic soil pressures are included in the design of basement and retaining walls. Because the requirement can be onerous for small structures and retaining walls, the applicability is limited to those walls that are higher than 6 feet (1.83 m). Section 7.5.1 of the 1997 NEHRP Provisions commentary includes a discussion about how earth-retaining structures have been designed for dynamic loads.

Additionally, a thorough assessment of potential consequences of any liquefaction and soil strength loss needs to be made and considered in the design of the structure. See the commentary to Section 1803.5.11 for a discussion of earthquake damage.
due to liquefaction. Design to mitigate damage due to liquefaction consists of three parts: evaluation of liquefaction hazard; evaluation of potential ground displacement and designing to resist ground displacement, reducing the potential for liquefaction or choosing an alternative site with a lower hazard. The assessment is required to be made for the site's peak ground accelerations that are consistent with the maximum considered earthquake (MCE) ground motions. In this way the potential for liquefaction as well as the effects of liquefaction during the MCE are considered in design. This is consistent with the risk-based targets for collapse prevention as a performance goal and other evaluations for the MCE.

1803.6 Reporting. Where geotechnical investigations are required, a written report of the investigations shall be submitted to the building official by the owner or authorized agent at the time of permit application. This geotechnical report shall include, but need not be limited to, the following information:

1. A plot showing the location of the soil investigations.
2. A complete record of the soil boring and penetration test logs and soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered.
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
7. Deep foundation information in accordance with Section 1803.5.5.
8. Special design and construction provisions for foundations of structures founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section 1803.5.8.
10. Controlled low-strength material properties and testing in accordance with Section 1803.5.9.

If a written report is required by the building official, it is required to include at a minimum the items listed in this section. These items will establish a retrievable and verifiable record of the soil conditions if problems are encountered in the future. These items also provide the minimum necessary information for compliance with the code and an adequate foundation system.

SECTION 1804
EXCAVATION, GRADING AND FILL

1804.1 Excavation near foundations. Excavation for any purpose shall not remove lateral support from any foundation without first underpinning or protecting the foundation against settlement or lateral translation.

The purpose of this section is to provide for stability of adjacent foundations when excavations are made. The method to be used, whether it be shoring or underpinning, must be addressed by a geotechnical investigation as required in Section 1803.5.7. Due to their lack of shear strength, cohesionless soils, such as sand, will slide to the bottom of an excavation until a certain slope of the sides is reached. This slope is known as the angle of repose of natural slope and is independent of the depth of the excavation.

Cohesive (fine-grained) soils, such as clay, behave much differently compared to granular materials. For example, unsupported vertical cuts of 20 feet (6096 mm) or more can be made in stiff plastic clay materials. This is due to a firm bond between the particles of the cohesive soil. But the strength of this bond (cohesiveness) will vary based on the conditions of the soil, such as density, water content, plasticity and sensitivity (loss of shear strength upon disturbance).

In cohesive soils, when a certain critical depth of excavation is reached, the sides of the cut will fail and the soil mass will fall to the bottom. Unlike granular materials, such as sand, the steepest slope at which a cohesive soil will stand decreases as the depth of the excavation increases. Technically, in cohesive soils, the resistance against sliding is a function of the shearing resistance of the material and its corresponding angle of internal friction (frictional resistance between particles).

The use of the angle of internal friction (and other factors) to calculate slope stability is applicable not only to cohesive soils, but also to granular materials. For example, when the slope angle of an excavation in a bed of sand exceeds the angle of internal friction of the material, the sand will slide down the slope; therefore, the steepest slope that sand can attain is equal to the angle of internal friction. The angle of repose (previously discussed) will be approximately the same value as the angle of internal friction only when the sand is in a dry and loose condition (such as in a stockpile) or is fully immersed in water.

For simplicity, what we have been dealing with in this part of the commentary is slope stability as it relates to homogeneous soils, such as sand and clay. In nature, however, soils often occur as mixtures or layers (strata) of different materials, making the determination of slope stability a highly technical and complex subject. Normally for shallow excavations, determination of safe slopes is a matter of applying local experience. In cases of deep cuts, however, slope stability is best determined through tests and analytical methods performed by professionals experienced in foundation engineering.

Some texts dealing with soil mechanics contain tables that indicate the angles of slopes that can be expected for various soil materials commonly found throughout the country. While such tables provide
useful information, they should be used only as a guide. They should not be used for design purposes, nor employed when a critical subsurface condition exists or when the type of soil (established by borings) does not closely fit those described in the tables.

1804.2 Placement of backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or with a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted in a manner that does not damage the foundation or the waterproofing or dampproofing material.

Exception: CLSM need not be compacted.

This section requires that soils used for backfilling foundation excavations must be free of organic material, construction debris or large rocks. The type of soil used for backfill purposes becomes an important consideration in the design of foundation walls. For example, clean sand, gravel or a mixture of these two granular materials is considered the best kind of backfill to use because each is free draining and generally has frost-free properties. On the other hand, fine-grained soils, such as clays, tend to accumulate moisture and are susceptible to swelling and shrinking, as well as frost action. Such backfill materials, particularly at times when shrinkage cracks occur, can become loaded with rainwater, thus subjecting foundation walls and basement floors to hydrostatic pressures and possible structural damage.

Backfilling and related work should be performed in such a way as to prevent the movement of the earth of adjoining properties or the subsequent caving in of backfilled areas. Backfilling should not be done until retaining walls, foundation walls or other construction against which backfill is to be placed is in a suitable condition to resist lateral pressures. This section requires that backfill be free of organic materials, construction debris, cobbles and boulders.

In addition to carefully selecting the backfill material, the soil should be placed in lifts, usually 9 inches (229 mm) or less, and compacted to prevent significant subsidence due to consolidation under its own weight. While compaction is done by hand-operated tampers or other portable compaction equipment, care should be taken to prevent any possible damage to waterproofing or dampproofing installations and to avoid overcompaction of backfill since it may cause excessive earth pressure against foundation walls. As an alternative to compacted backfill, the code also permits CLSM (see commentary, Section 1804.6).

1804.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation shall be permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

This section requires that the ground immediately adjacent to the foundation be sloped away from the building. The intent is to facilitate water drainage and reduces the potential for water standing under and around the building.

Where a full 10 feet (3048 mm) of slope is not available on a building site, an alternative method of diverting water away from the foundation is permitted by providing a minimum 5-percent slope to an approved diversion structure. The use of swales to convey surface water is recognized, provided the minimum slope is provided where necessary.

The exception permits the slope to be reduced to a rate of 1 unit vertical in 48 units horizontal (2-percent slope) where climatic or soil conditions warrant. This exception would be applicable in arid areas or sites that are surrounded by free-draining soils, such as sand.

1804.4 Grading and fill in flood hazard areas. In flood hazard areas established in Section 1612.3, grading and/or fill shall not be approved:

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.

2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a registered design professional in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the design flood.

3. In flood hazard areas subject to high-velocity wave action, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.

4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed flood hazard area encroachment, when combined with all other existing and anticipated flood hazard area encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

This section puts limitations on fill and grading in flood hazard areas. In many flood hazard areas it is common to use fill to achieve the appropriate elevation of the building's lowest floor. Item 1 intends to
SOILS AND FOUNDATIONS

minimize the risk of fills becoming unstable in a flood. Fill materials that become saturated during conditions of flooding may become unstable and fill slopes may be exposed to erosive velocities and waves. When placed in a flood hazard area, fill and grading must remain stable as floodwaters rise and, in particular, as floodwaters fall and the saturated materials drain.

Item 2 permits grading or fill in a floodway only if it is demonstrated that it will not adversely affect surrounding areas by increasing the design flood elevation. As the definition indicates, a “Floodway” is that portion of flood hazard areas along rivers and streams that must be reserved for the discharge of the design flood event. The National Flood Insurance Program (NFIP) requires that the impact of development or encroachment into the floodway must be considered.

In flood hazard areas that are subject to high-velocity wave action, fill can divert the flow of floodwaters and increase flood risks on other properties. Item 3 provides coordination with ASCE 24 provisions relating to fill in flood hazard areas subject to high-velocity wave action. These areas are commonly referred to as “coastal high hazard areas” or “V zones,” and typically are indicated on flood hazard maps of communities along the open coast. In these areas, changing the shape of the ground through grading or fill can divert erosive flows and increase wave energies that, in turn, increase the forces that affect a building and any adjacent structures. It is also notable that ASCE 24 specifies that fill may not be used for structural support of buildings in these areas.

Although the Federal Emergency Management Agency (FEMA) designates floodways on many rivers and streams shown on Flood Insurance Rate Maps (FIRMs), some riverine flood hazard areas have base flood elevations, but do not have delineated floodways. In these areas, the effect of floodplain development on flood elevations has not been evaluated. Item 4 provides consistency with NFIP with respect to development in areas where a base flood elevation has been established, but no floodway has been designated. It allows a proposed development to be approved if it is demonstrated that the flood elevations will not be increased by more than 1 foot (305 mm). Development in riverine flood plains can increase flood levels and loads on other properties, particularly if it occurs in areas considered floodways that must be reserved to convey flood flows (see the definition of "Floodway").

FEMA TB #10, Ensuring That Structures Built on Fill In or Near Special Flood Hazard Areas Are Reasonably Safe From Flooding, offers guidance to communities to determine whether structures on fill are reasonably safe from flooding. This determination is required by FEMA if property owners submit docu-

mentation requesting that FEMA show that filled land is (or will be) no longer subject to flooding by the base flood. If the documentation meets FEMA’s requirements, a Letter of Map Revision based on Fill (LOMR-F) is issued to revise the FIRM.

1804.5 Compacted fill material. Where shallow foundations will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved geotechnical report, as set forth in Section 1803.

Exception: Compacted fill material 12 inches (305 mm) in depth or less need not comply with an approved report, provided the in-place dry density is not less than 90 percent of the maximum dry density at optimum moisture content determined in accordance with ASTM D 1557. The compaction shall be verified by special inspection in accordance with Section 1705.6.

Where prepared fill is to be utilized for foundation support, the geotechnical report is to contain detailed information for approval of the fill. The information on the prepared fill in the geotechnical report is necessary to permit a reasonable prediction as to the load-bearing capacity of the fill material. Additionally, when prepared fill is to be utilized where special inspections are required, the fill operation itself must be performed under the scrutiny of a special inspector. The exception permits a limited depth of fill to be placed in accordance with prescriptive criteria rather than requiring details of fill placement in a geotechnical report. Special inspection is a requirement.

1804.6 Controlled low-strength material (CLSM). Where shallow foundations will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved geotechnical report, as set forth in Section 1803.

As an alternative to compacted fill in accordance with Section 1804.5, this section permits the use of CLSM for the support of footings. CLSM must be placed in accordance with an approved report that includes requirements for the material strength and field verification. In Chapter 2, CLSM is defined as “self-compacting cementitious materials.” This class of material is commonly referred to by a variety of other names, which include flowable fill, controlled density fill, unshrinkable fill and soil-cement slurry. Guidance on the use of these materials can be found in ACI 229R (see Section 1803.5.9).

SECTION 1805
DAMPPROOFING AND WATERPROOFING

1805.1 General. Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed and dampproofed in accordance with this section, with the exception of those spaces containing groups
other than residential and institutional where such omission is not detrimental to the building or occupancy.

Ventilation for crawl spaces shall comply with Section 1203.4.

Section 1805 covers the requirements for waterproofing and dampproofing those parts of substructure construction that need to be provided with moisture protection. It identifies the locations where moisture barriers are required and specifies the materials to be used and the methods of application. The provisions also deal with subsurface water conditions, drainage systems and other protection requirements.

The term "waterproofing" is at times used where dampproofing is the minimum requirement. Although both terms are intended to apply to the installation and use of moisture barriers, dampproofing does not furnish the same degree of protection against moisture.

Dampproofing generally refers to the application of one or more coatings of a compound or other materials that are impervious to water, which are used to prevent the passage of water vapor through walls or other building components, and which restrict the flow of water under slight hydrostatic pressure. Waterproofing, on the other hand, refers to the application of coatings and sealing materials to walls or other building components to prevent moisture from penetrating in either a vapor or liquid form, even under conditions of significant hydrostatic pressure. Hydrostatic pressure is created by the presence of water under pressure. This pressure can occur when the ground-water table rises above the bottom of the foundation wall, or the soil next to the foundation wall becomes saturated with water caused by uncontrolled storm water runoff.

Section 1805.1 is an overall requirement that waterproofing and dampproofing applications are to be made to horizontal and vertical surfaces of below-ground spaces where the occupancy would normally be affected by the intrusion of water or moisture. Moisture or water in a floor below grade can cause damage to structural members, such as columns, posts or load-bearing walls, as well as pose a health hazard by promoting the growth of bacteria or fungi and adversely affect any mechanical and electrical appliances that may be located at that level. It can also cause a great deal of damage to goods that may be located or stored in that lower level. These vertical and horizontal surfaces include foundation walls, retaining walls, underfloor spaces, and floor slabs. Waterproofing and dampproofing are not required in locations other than residential and institutional occupancies where the omission of moisture barriers would not adversely affect the use of the spaces. An example of a location where waterproofing or dampproofing would not be required is in an open parking structure, as long as the structural components are individually protected against the effects of water. Waterproofing and dampproofing are not permitted to be omitted from residential and institutional occupancies where people may be sleeping or services are provided on the floor below grade. A person walking in a flooded basement may be in a very hazardous situation, particularly if the possibility of an electrical charge in the water exists that is caused by electrical service at that level.

Section 1805.1.1 addresses the type of problem faced when a portion of a story is above grade, while Section 1805.1.2 limits any infiltration of water into crawl spaces so as to protect this area from potential water damage and prevent ponding of water. Both of these sections reference other applicable sections of the code, as well as the exceptions.

1805.1.1 Story above grade plane. Where a basement is considered a story above grade plane and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be dampproofed in accordance with Section 1805.2 and a foundation drain shall be installed in accordance with Section 1805.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections 1803.5.4, 1805.3 and 1805.4.1 shall not apply in this case.

The provisions of this section, stated in another way, require that where a basement is deemed to be a story above grade plane (see definition, Section 202), the section of the basement floor that occurs below the exterior ground level and the walls that bound that part of the floor are to be dampproofed in accordance with the requirements of Section 1805.2.

The use of dampproofing, rather than waterproofing, is permitted here since hydrostatic pressure will not tend to develop against the walls if the basement is a story above grade plane and the ground level adjacent to the basement wall is below the basement floor elevation for no less than 25 percent of the basement perimeter.

Any water pressure that may occur against the walls below ground or under the basement floor would be relieved by the water drainage system required in this section. The drainage system would be installed at the base of the wall construction in accordance with Section 1805.4.2 for a minimum distance along those portions of the wall perimeter where the basement floor is below ground level. Because of the relationship of grade to the basement floor and the inclusion of foundation drains, the potential for hydrostatic pressure buildup is not significant; therefore, a ground-water table investigation, waterproofing and the basement floor gravel base course is not required.

The objective of Section 1805.4.1 is to prevent moisture migration in basement spaces. In story-above-grade construction that meets the requirements of this section, the basement floor would be only partly below ground level (sometimes a small part) and the need for moisture protection as required by Section 1805.4.1 would be unnecessary. Damp-
SOILS AND FOUNDATIONS

proofing of the floor slab would be required, however, in accordance with Section 1805.2.1.

1805.1.2 Under-floor space. The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the ground-water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 1803.5.4, 1805.2, 1805.3 and 1805.4 shall not apply in this case.

- The requirements of this section are designed to prevent any ponding of water in underfloor spaces, such as crawl spaces. Crawl spaces are particularly susceptible to ponding of water, since they are usually uninhabitable spaces that are observed very infrequently. Water can build up in these spaces and remain for an extended period of time without being noticed by the building occupants. This type of stagnant water under a building, which can harbor disease, mold and disease-carrying insects, such as mosquitoes, can result in a serious health concern. Water buildup in a crawl space can also damage the structural integrity of the building. Wood exposed to water will deteriorate and rot, while concrete and masonry exposed to water will deteriorate with a loss of strength.

Steel exposed to water or high humidity can eventually rust to the extent that effective structural capability is jeopardized. Water buildup in a crawl space can also damage any mechanical or electrical appliances, which may be located in the space, by causing corrosion of electrical parts or metal skins and deterioration to insulation used to protect heating elements.

Where it is known that the water table can rise to within 6 inches (152 mm) of the outside ground level, or where there is evidence that surface water cannot readily drain from the site, then the finished ground surface in underfloor spaces is to be set at an elevation equal to the outside ground level around the perimeter of the building unless an approved drainage system is provided. In order for the drainage system to be approved, it must be demonstrated to be adequate to prevent the infiltration of water into the underfloor space. This is done by determining the maximum possible flow of water near the foundation wall and footing and designing the drainage system to remove that flow of water as it occurs, without permitting the buildup of water at the foundation wall.

To prevent the ponding of water in the underfloor space from a rise in the ground-water table, or from storm water runoff, the finished ground level of an underfloor space is not to be located below the bottom of the foundation footings.

Damproofing (see Section 1805.2) the foundation walls, waterproofing (see Section 1805.3) and providing subsoil drainage (see Section 1805.4) is not necessary if the ground level of the underfloor space is as high as the ground level at the outside of the building perimeter, as the foundation walls do not enclose an interior space below grade. Compliance with Sections 1805.2, 1805.3 and 1805.4 would still be required where the finished ground surface of the underfloor space is below the outside ground level.

1805.1.2.1 Flood hazard areas. For buildings and structures in flood hazard areas as established in Section 1612.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level on at least one side.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/FIA-TB-11.

- The definition of "Basement" for buildings and structures located in flood hazard areas is "the portion of a building having its floor subgrade (below grade) on all sides." This definition pertains to enclosed areas below elevated buildings and structures whether or not there is enough clearance for such areas to be occupied, such as a crawl space. Neither the use of the enclosed space nor the clear height is the deciding factor as to whether an enclosed area below an elevated building is considered a basement under Section 1612.2. The controlling factor is whether the interior grade is below the exterior grade on all sides, even if the interior grade is established by the footing excavation that was not backfilled.

Enclosed areas below elevated buildings or structures must meet the requirements of Section 1612.4, which references ASCE 24. In the event that a building or structure is not built in accordance with these requirements, particularly if it is determined to have a basement that is subgrade on all sides, NFIP flood insurance premium rates may be significantly higher.

In recognition of common construction practices in some parts of the country, the exception permits crawl spaces to be as much as 2 feet (610 mm) below the lowest adjacent exterior grade in accordance with FEMA Technical Bulletin #11, provided the additional requirements of that document are met (specifically, the maximum depth below grade is limited to 2 feet (210 mm) and maximum depth of floodwater above grade is limited to 2 feet (210 mm)). Communities that choose to allow this practice must amend their ordinances to include these additional requirements.

1805.1.3 Ground-water control. Where the ground-water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section 1805.2. The design of the system to lower the ground-water table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, per-
meability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to operate and the rated capacity of the disposal area of the system.

- After completion of building construction, it is often necessary to maintain the water table at a level that is at least 6 inches (152 mm) below the bottom of the lowest floor in order to prevent the flow or seepage of water into the basement. Where the site consists of well-draining soil and the highest point of the water table occurs naturally at or lower than the required level stated above, there is no need to provide a site drainage system specifically designated to control the ground-water level. Where the soil characteristics and site topography are such that the water table can rise to a level that will produce a hydrostatic pressure against the basement structure, then a site drainage system may be installed to reduce the water level if there is sufficient land area to accomplish the purpose. When ground-water control in accordance with this section is provided, waterproofing in accordance with Section 1805.3 is not required.

There are many types of site drainage systems that can be employed to control ground-water levels. The most commonly used systems may involve the installation of drainage ditches or trenches filled with pervious materials, sump pits and discharge pumps, well point systems, drainage wells with deep-well pumps, sand-drain installations, etc. This section requires that all such systems be designed and constructed using accepted engineering principles and practices based upon considerations that include the permeability of the soil, amount and rate at which water enters the system, pump capacity, capacity of the disposal area and other such factors that are necessary for the complete design of an operable drainage system.

1805.2 Dampproofing. Where hydrostatic pressure will not occur as determined by Section 1803.5.4, floors and walls for other than wood foundation systems shall be dampproofed in accordance with this section. Wood foundation systems shall be constructed in accordance with AF&PA PWF.

- For a general definition of "Dampproofing," see the commentary to Section 1805.1. Where a ground-water table investigation made in accordance with the requirements of Section 1803.5.4 (see commentary) has established that the high water table will occur at such a level that the building substructure will not be subjected to hydrostatic pressure, then dampproofing in accordance with this section and a subsoil drain in accordance with Section 1805.4 are sufficient to control moisture in the floor below grade. Since the wall will not be subject to water under pressure, the more restrictive provisions of waterproofing, as outlined in Section 1805.3, are not required. Wood foundation systems specified in Section 1807.1.4 (see commentary) are to be dampproofed as required by the American Forest & Paper Association (AF&PA) Permanent Wood Foundation Design Specification (PWF).

Dampproofing products having ICC Evaluation Service reports can be viewed at www.icc-es.org.

1805.2.1 Floors. Dampproofing materials for floors shall be installed between the floor and the base course required by Section 1805.4.1, except where a separate floor is provided above a concrete slab.

Where installed beneath the slab, dampproofing shall consist of not less than 6-mil (0.006 inch; 0.152 mm) polyethylene with joints lapped not less than 6 inches (152 mm), or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 4-mil (0.004 inch; 0.102 mm) polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

- Floors requiring dampproofing in accordance with Section 1805.2 are to employ materials as specified in Section 1805.2.1. The dampproofing materials must be placed between the floor construction and the supporting gravel or stone base as shown in Figure 1804.4.2. Even if a floor base in accordance with Section 1805.4.1 is not required, dampproofing is still to be placed under the slab unless otherwise specified in Section 1805.2.1.

The installation is intended to provide a moisture barrier against the passage of water vapor or seepage into below-ground spaces.

The dampproofing material most commonly used for underslab installations consists of a polyethylene film no less than 6 mil [0.006 inch; (0.152 mm)] in thickness, which is applied over the gravel or stone base required in Section 1805.4.1. Care must be used in the installation of the material over the rough surface of the base and during the concreting operations so as not to puncture the polyethylene. Joints must be lapped at least 6 inches (152 mm). Other materials used in a similar way are made of neoprene or butyl rubber. Dampproofing materials can also be applied on top of the base concrete slab if a separate floor is provided above the base slab, since dampproofing is provided to prevent moisture infiltration of the interior space, not the concrete slab.

Materials commonly used for dampproofing floors are listed in Figure 1805.2.2.

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td>ASTM D 449</td>
</tr>
<tr>
<td>Asphalt primer</td>
<td>ASTM D 41</td>
</tr>
<tr>
<td>Coal-tar</td>
<td>ASTM D 450</td>
</tr>
<tr>
<td>Concrete and masonry oil primer</td>
<td>ASTM D 43</td>
</tr>
<tr>
<td>(for coal-tar applications only)</td>
<td></td>
</tr>
<tr>
<td>Treated glass fabric</td>
<td>ASTM D 1668</td>
</tr>
</tbody>
</table>

Figure 1805.2.2
MATERIALS FOR WATERPROOFING AND DAMPPROOFING INSTALLATIONS
SOILS AND FOUNDATIONS

1805.2.2 Walls. Dampproofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level.

Dampproofing shall consist of a bituminous material, 3 pounds per square yard (16 N/m²) of acrylic modified cement, 1/4 inch (3.2 mm) coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section 1805.3.2 or other approved methods or materials.

- Walls requiring dampproofing in accordance with Section 1805.2 are first to be prepared as required in Section 1805.2.2.1 and then coated with a bituminous material, cement or mortar as specified in Section 1805.2.2 or other approved materials and methods of application. Approved materials are those that will prevent moisture from penetrating the foundation wall when water is present but not under pressure.

- Coatings are applied to cover prepared exterior wall surfaces extending from the top of the wall footings to slightly above ground level so that the entire wall that contacts the ground is protected. Surfaces are usually primed to provide a bond coat and then dampproofed with a protective coat of asphalt or tar pitch. Emulsion-type coatings may be applied directly on "green" concrete or unit masonry walls; however, because they are water-soluble materials, their use is not generally recommended. Installation should comply with the manufacturer's instructions.

- Any of the materials specified in Section 1805.3.2 for waterproofing are also allowed to be used for dampproofing. Figure 1805.2.2 provides a list of bituminous materials that can be used, including the applicable standards that may be used as the basis of acceptance of such materials. Included in Figure 1805.2.2 is ASTM D 1668 for glass fabric that is treated with asphalt (Type I), coal-tar pitch (Type II) or organic resin (Type III).

- Surface-bonding mortar complying with ASTM C 887 may be utilized. This specification covers the materials, properties and packaging of dry, combined materials for use as surface-bonding mortar with concrete masonry units that have not been prefaced, coated or painted. Since this specification does not address design or application, manufacturers' recommendations should be followed. This standard covers proportioning, physical requirements, sampling and testing. The minimum thickness of the coating is 1/8 inch (3.2 mm).

- Acrylic-modified cement coatings may be utilized at the rate of 3 pounds per square yard (16 N/m²). These types of materials have been used and performed successfully as dampproofing materials for foundation walls. Surface-bonding mortar and acrylic-modified cement are limited in use to dampproofing. The ability of these two types of products to bridge nonstructural cracks, as required in Section 1805.3.2 for waterproofing materials, is not known; therefore, their use is limited to dampproofing and they are not permitted to be used as waterproofing. Dampproofing may also include other materials and methods of installation acceptable to the building official.

1805.2.2.1 Surface preparation of walls. Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface below ground level with not less than 1/4 inch (9.5 mm) of Portland cement mortar. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

- Before applying dampproofing materials, the concrete must be free of any holes or recesses that could affect the proper sealing of the wall surfaces. Air trapped beneath a dampproofing coating or membranes can cause blistering, while rocks and other sharp objects can puncture membranes. Irregular surfaces can also create uneven layering of coatings, which can result in vulnerable areas of dampproofing. Surface irregularities commonly associated with concrete wall construction can be sealed with bituminous materials or filled with portland cement grout or other approved methods.

- Unit masonry walls are usually parged (plastered) with a 1/8-inch-thick (12.7 mm) layer of portland cement and sand mix (1:2 1/2 by volume) or with Type M mortar proportioned in accordance with the requirements of ASTM C 270, and applied in two 1/4-inch-thick (6.4 mm) layers. In no case is parging to result in a final thickness of less than 1/8 inch (9.5 mm). The parging is to be coved at the joint formed by the base of the wall and the top of the wall footing to prevent the accumulation of water at that location. The moisture protection of unit masonry walls provided by the parging method may not be required where approved dampproofing materials, such as grout coatings, cement-based paints or bituminous coatings, can be applied directly to masonry surfaces.

1805.3 Waterproofing. Where the ground-water investigation required by Section 1803.5.4 indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section 1805.1.3, walls and floors shall be waterproofed in accordance with this section.

- For a general definition of "Waterproofing," and the distinction between dampproofing and waterproofing, see the commentary to Section 1805.1.

- The significance of waterproofing installations is that they are intended to provide moisture barriers against water seepage that may be forced into below-ground spaces by hydrostatic pressure.

- Where a ground-water table investigation made in accordance with the requirements of Section 1803.5.4 (see commentary) has established that the
high water table will occur at such a level that
the building substructure will be subjected to hydrostatic
pressure, and where the water table is not lowered by
a water control system, as prescribed in the com-
mentary to Section 1805.1.3, all floors and walls below
ground level are to be waterproofed in accordance
with Sections 1805.3.1, 1805.3.2 and 1805.3.3.
Waterproofing products having ICC Evaluation Ser-
vice reports can be viewed at www.icc-es.org.

1805.3.1 Floors. Floors required to be waterproofed shall be
of concrete and designed and constructed to withstand
the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a mem-
brane of rubberized asphalt, butyl rubber, fully adhered/fully
bonded HDPE or polyolefin composite membrane or not less
than 6-mil [0.006 inch (0.152 mm)] polyvinyl chloride with
joints lapped not less than 6 inches (152 mm) or other
approved materials under the slab. Joints in the membrane
shall be lapped and sealed in accordance with the manufac-
turer’s installation instructions.

Since floors that are required to be waterproofed are
subjected to hydrostatic uplift pressures, such floors
must, for all practical purposes, be made of concrete and
designed and constructed to resist the maximum
hydrostatic pressures possible. It is particularly
important that the floor slab be properly designed,
since severe cracking or movement of the concrete
would allow water seepage into below-ground spaces
because the ability of the waterproofing materials to
bridge small cracks would be exceeded. Concrete
floor construction is to comply with the applicable pro-
visions of Chapter 19.

Below-ground floors subjected to hydrostatic uplift
pressures are to be waterproofed with membrane materials
placed as underslab or split-slab installations,
including such materials as rubberized asphalt,
butyl rubber, fully bonded High Density Polyethylene
(HDPE), fully bonded polyolefin and neoprene or with
polyvinyl chloride (PVC) not less than 6 mil [0.006
inch; (0.152 mm)] in thickness, lapped at least 6
inches (152 mm). There are many proprietary mem-
brane products available in the marketplace that are
specifically made for waterproofing floors and walls
(i.e., polyethylene sheets sandwiched between layers
of asphalt), which may be used for that purpose when
approved by the building official.

All membrane joints are to be lapped and sealed in
accordance with the manufacturer’s instructions to
form a continuous, impermeable moisture barrier.

1805.3.2 Walls. Walls required to be waterproofed shall be of
concrete or masonry and shall be designed and constructed to
withstand the hydrostatic pressures and other lateral loads to
which the walls will be subjected.

Waterproofing shall be applied from the bottom of the
wall to not less than 12 inches (305 mm) above the maximum
elevation of the ground-water table. The remainder of
the wall shall be dampproofed in accordance with Section
1805.2.2. Waterproofing shall consist of two-ply hot-mopped
felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl
chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified
asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene or other
approved methods or materials capable of bridging nonstruc-
tural cracks. Joints in the membrane shall be lapped and
sealed in accordance with the manufacturer’s installation
instructions.

Walls that are required to be waterproofed in accor-
dance with Section 1805.3 must first be prepared as
required in Section 1805.3.2.1.

The walls must be designed to resist the hydro-
static pressure anticipated at the site, as well as any
other lateral loads to which the wall will be subjected,
such as soil pressures or seismic loads. As with
floors required to be waterproofed, it is particularly
important that walls required to be waterproofed be
properly designed to resist all loads present, since
cracking and other damage would allow water seep-
age into below-ground spaces. Water seepage can
lead to deterioration of the foundation as wood rots,
cracks and masonry erode and steel rusts. Such
deterioration can cause structural failure of the foun-
dation. More importantly, failure of the foundation wall
can lead to structural failure of the building, since the
foundation supports the building structure. Masonry or
cement construction must comply with the appli-
cable provisions of Chapters 21 and 19, respectively.

Figures 1805.2.2 and 1805.3.2 list materials com-
monly used for the installation of moisture barriers in
wall construction and the related standard that may
be used as a basis for acceptance of such materials.
Asphalt and coal-tar products are not compatible and
should not be used together.

Waterproofing installations are to extend from the
bottom of the wall to a height no less than 12 inches
(305 mm) above the maximum elevation of the
ground-water table determined in accordance with
the requirements of Section 1803.5.4 (see com-
mentary). The remainder of the wall below ground level (if
the height is small) may be waterproofed as a contin-
uation of the installation or must be dampproofed in
accordance with the requirements of Section
1805.2.2. If the ground-water table investigation is

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt-saturated asbestos felt</td>
<td>ASTM D 250</td>
</tr>
<tr>
<td>Asphalt-saturated burlap fabric</td>
<td>ASTM D 1327</td>
</tr>
<tr>
<td>Asphalt-saturated cotton fabric</td>
<td>ASTM D 173</td>
</tr>
<tr>
<td>Asphalt-saturated organic felt</td>
<td>ASTM D 226</td>
</tr>
<tr>
<td>Coal-tar-saturated burlap fabric</td>
<td>ASTM D 1327</td>
</tr>
<tr>
<td>Coal-tar-saturated cotton fabric</td>
<td>ASTM D 173</td>
</tr>
<tr>
<td>Coal-tar-saturated organic felt</td>
<td>ASTM D 227</td>
</tr>
</tbody>
</table>

Figure 1805.3.2
MATERIALS FOR
WATERPROOFING INSTALLATIONS
not conducted on the basis of the exception to Section 1803.5.4, then Waterproofing should be provided from a point below the footing to above the ground level.

This section requires that waterproofing must consist of two-ply hot-mopped felts. The practice of the waterproofing industry is to select the number of plies of membrane material based on the hydrostatic head (height of water pressure against the wall). As a general practice, if the head of water is between 1 foot (305 mm) and 3 feet (914 mm), two plies of felt or fabric membrane are used; between 4 feet (1219 mm) and 10 feet (3048 mm), three-ply construction is needed and between 11 feet (3353 mm) and 25 feet (7620 mm), four-ply construction is necessary.

Waterproofing installations may also use polyvinyl chloride materials of no less than 6-mil [0.006 inch (0.152 mm)] thick, 40-mil [0.040 inch (1.02 mm)] polymer-modified asphalt or 6-mil [0.006 inch (0.152 mm)] polyethylene. These materials have been widely recognized for their effectiveness in bridging nonstructural cracks. Other approved materials and methods may be used provided that the same performance standards are met. All membrane joints must be lapped and sealed in accordance with the manufacturer's instructions.

1805.3.2.1 Surface preparation of walls. Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 1805.2.2.1.

- Before applying waterproofing materials to concrete or masonry walls, the surfaces must be prepared in accordance with the requirements of Section 1805.2.2.1, which requires the sealing of all holes and recesses. Surfaces to be waterproofed must also be free of any projections that might puncture or tear membrane materials that are applied over the surfaces.

1805.3.3 Joints and penetrations. Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water-tight utilizing approved methods and materials.

- This section requires that all joints occurring in floors and walls and at locations where floors and walls meet, as well as all penetrations in floors and walls, be made water tight by approved methods. Sealing joints and penetrations is of primary importance to ensuring the effectiveness of the waterproofing. If the joints or penetrations are not sealed properly, they can develop leaks, which become a passageway for water to enter the building. Since the remainder of the foundation is wrapped in waterproofing, moisture can actually become trapped in the foundation walls or floor slab, and serious damage to these structural components can occur. Such methods may involve the use of construction keys (e.g., between the base of the wall and the top of the footing) or, if there is hydrostatic pressure, floor and wall joints may require the use of manufactured waterstops made of metal, rubber, plastic or mastic materials. Figures 1805.3.3(1) and 1805.3.3(2) illustrate examples of joint treatment and penetration treatment of waterproofing. Floor edges along walls and floor expansion joints may employ the use of any number of preformed expansion joint materials, such as asphalt, polyurethane, sponge rubber, self-expanding cork, cellular fibers bonded with bituminous materials, etc., which all comply with applicable ASTM International (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) standards or other federal specifications. A variety of sealants may be used together with the preformed joint materials. Gaskets made of neoprene and other materials are also available for use in concrete and masonry joints. The National Roofing Contractors Association's (NRCA's) Roofing and Waterproofing Manual pro-

![Figure 1805.3.3(1)]
**Figure 1805.3.3(1)**
DETAIL—MEMBRANE PLACEMENT THROUGH A KEY JOINT
vides details for the reinforcement of membrane terminations, corners, intersections of slabs and walls, through-wall and slab penetrations and other locations.

Penetrations in walls and floors may be made water-tight with grout or manufactured fill materials and sealants made for that purpose.

1805.4 Subsoil drainage system. Where a hydrostatic pressure condition does not exist, dampproofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section 1805.1.3 shall be deemed adequate for lowering the groundwater table.

- This section covers subsoil drainage systems in conjunction with damp proofing (see Section 1805.2) to protect below-ground spaces from water seepage. Such systems are not used where basements or other below-ground spaces are subject to hydrostatic pressure because they would not be effective in disposing of the amount of water anticipated if a hydrostatic pressure condition exists. Ground-water tables may be reduced to acceptable levels by methods described in the commentary to Section 1805.1.3.

The details of subsoil drainage systems are covered in the requirements of Sections 1805.4.1 through 1805.4.3.

1805.4.1 Floor base course. Floors of basements, except as provided for in Section 1805.1.1, shall be placed over a floor base course not less than 4 inches (102 mm) in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

- This section requires that basement floors, except for story-above-grade construction, must be placed on a gravel or stone base no less than 4 inches (102 mm) thick. Not more than 10 percent of the material is to pass a No. 4 sieve so as to provide a porous installation. Material that passes a No. 4 sieve would be fine silt or clay that would not permit the free movement of water through the floor base. This requirement serves three purposes. The first is to provide an adjustment to the irregularities of a compacted subgrade so as to produce a level surface upon which to cast a concrete slab. The second is to provide a capillary break so that moisture from the soil below will not rise to the underside of the floor. Finally, where required, the porous base can act as a drainage system to expel underslab water by means of gravity or the use of a sump pump or other approved methods.

The exception allows for the omission of the floor base when the natural soils beneath the floor slab consist of well-draining granular materials, such as sand, stone or mixtures of these materials. The exception is consistent with the requirements of Section 1805.4.3. Some caution, however, is justified in the use of this exception. If the granular soils contain an excessive percentage of fine materials, the porosity and the ability of the soil to provide a capillary break may be considerably diminished. The exception is only to be applied if the natural base is equivalent to the floor base otherwise required by this section.

1805.4.2 Foundation drain. A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the
invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section 1805.4.1, and shall be covered with not less than 6 inches (152 mm) of the same material.

This section describes the materials and features of construction required for the installation of foundation drain systems. Perimeter foundation drains provide a means to remove free ground water and prevent leakage into habitable below-grade spaces. The first part of this provision describes a foundation drain of gravel or crushed stone. The balance of the provision describes the installation of drain tile or perforated pipe where either of those is provided. The use of drain tiles is important in areas having moderate to heavy rainfall as well as in soils that have low percolation rates. The requirements of this section are illustrated in Figure 1805.4.2.

This type of drain system is suitable where the water table occurs at such elevation that there is no hydrostatic pressure exerted against the basement floor and walls, and where the amount of seepage from the surrounding soil is so small that the water can be readily discharged by gravity or mechanical means into sewers or ditches. The objective is to combine the protection afforded by the dampproofing of walls and floors (see Section 1805.2) and that given by perimeter drains so as to maintain below-ground spaces in a dry condition.

Gravel or crushed stone drain material is to be placed in the excavation so that it will extend out from the edge of the wall footing a distance of at least 12 inches (305 mm), with the bottom of the fill being no higher than the bottom of the base under the floor (see Section 1805.4.1) and the top of the filter material being no less than 6 inches (152 mm) above the top of the wall footing so that water will not collect along the top of the footing. Requiring the bottom of the gravel or crushed rock drain to be no higher than the bottom of the floor base is necessary so that if the water table rises into the floor base, it will also be able to rise unobstructed into the foundation drain. The foundation drain will then drain the water away from the building, as required by Section 1805.4.3. The top of the drain must be covered with an approved filter membrane to allow water to pass through without allowing, or at least greatly reducing, the possibility of fine soil materials entering the drainage system.

Where drain tile or perforated drain pipe is utilized,

![Diagram of foundation drainage system](image_url)

Figure 1805.4.2
FOUNDATION DRAINAGE SYSTEM

For SI: 1 inch = 25.4 mm.
the drain system consists of clay or concrete drain tiles or corrugated metal or nonmetallic pipes installed in a filter bed with at least 2 inches (51 mm) of filter material and covered with at least 6 inches (152 mm) of filter material to maintain good water flow into the drain tile or pipe. The foundation drain is set adjacent to the wall footing and extends around the perimeter of the building. Drain tiles are placed end to end with open joints to permit water to enter the system. Metallic and nonmetallic drains are made with perforations at the invert (bottom) section of the pipe and are installed with connected ends. The drain pipe invert is not to be set higher than the basement floor line such that water conveyed by the drain does not seep into the filter material and create a hydrostatic pressure condition against the foundation wall and footing. The inverts should not be placed below the bottom of the adjacent wall footings so as to avoid carrying away fine soil particles that, in time, could undermine the footing and settlement of the foundation walls.

Tile joints or pipe perforations should be covered with an approved filter membrane material to prevent them from becoming clogged, preventing fine particles that may be contained in the surrounding soil from entering the system and being carried away by water. The filter material around the drain tiles or pipes (not to be confused with filter membrane material) is to consist of selected gravel and crushed stone containing no more than 10 percent of material that passes through a No. 4 sieve. The filter materials should be selected to prevent the movement of particles from the protected soil surrounding the drain installation into the drain.

1805.4.3 Drainage discharge. The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the International Plumbing Code.

**Exception:** Where a site is located in well-drained gravel or sand/gravel mixture soils, a dedicated drainage system is not required.

This section references the International Plumbing Code® (IPC®) for the requirements of installing piping systems for the disposal of water from the floor base (see Section 1805.4.1) and the foundation drains (see Section 1805.4.2). Chapter 11 of the IPC deals with the piping materials, applicable standards and methods of installation of subsurface storm drains to facilitate water discharge either by gravity or mechanical means.

Where the soil at the site consists of well-drained granular materials (i.e., gravel or sand-gravel mixtures) to prevent the occurrence of hydrostatic pressure against the foundation walls and under the floor slab, the use of a dedicated drainage system as prescribed in the IPC is not required, since the site soils would permit natural drainage.

### SECTION 1806

**PRELIMINARY LOAD-BEARING VALUES OF SOILS**

1806.1 Load combinations. The presumptive load-bearing values provided in Table 1806.2 shall be used with the allowable stress design load combinations specified in Section 1605.3. The values of vertical foundation pressure and lateral bearing pressure given in Table 1806.2 shall be permitted to be increased by one-third where used with the alternative basic load combinations of Section 1605.3.2 that include wind or earthquake loads.

- This section clarifies the use of the presumptive load-bearing values relating to foundations and footing design. Foundation design is based on the ASD approach. Since Chapter 16 includes provisions and load combinations for strength design [load and resistance factor design (LRFD)], it is necessary to clarify that these values are to be used with the ASD load combinations.

1806.2 Presumptive load-bearing values. The load-bearing values used in design for supporting soils near the surface shall not exceed the values specified in Table 1806.2 unless data to substantiate the use of higher values are submitted and approved. Where the building official has reason to doubt the classification, strength or compressibility of the soil, the requirements of Section 1803.5.2 shall be satisfied.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

**Exception:** A presumptive load-bearing capacity shall be permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight or temporary structures.

Where the load-bearing capacity of the soil has not been determined by borings, as specified in Section 1803, the presumptive load-bearing values listed in Table 1806.2 are intended to apply in the design of shallow foundation systems.

While fill material (unconsolidated), mud, muck, peat, organic silt, soft clay and other unprepared fill materials are considered to have no presumptive load-bearing value, soil tests may show that they do have some limited load-bearing capacity and, based on this type of evidence, the building official may approve the construction of lightweight structures upon such soils.

The presumption is that the building official possesses sufficient technical knowledge on the character and behavior of subsurface materials to render a valid judgment on the adequacy of the soil to support satisfactorily the lightweight or temporary structure, or he or she has sought and gained specific advice.
through consultation with professionals who are competent in the field of foundation engineering. It would be an unwise practice to authorize construction on exceptionally weak soils without the benefit of technical knowledge to make judgmental decisions.

TABLE 1806.2. See below.

- The values in Table 1806.2 are intended to be lower-bound allowable pressures to be used where the bearing value is not determined by borings as specified in Section 1803. The classes of soil and rock listed in Table 1806.2 are those materials most commonly found at construction sites around the country. The allowable foundation pressures expressed in terms of pounds per square foot (psf) for each class of subsurface materials listed in Table 1806.2 are based on experience with the behavior of these materials under loaded conditions.

Should local soil conditions be significantly different than those listed in Table 1806.2, then, with the approval of the building official, local experience can be employed in the design of foundations, particularly where actual load-bearing values are less than the allowable unit pressures given in the table.

Whether the allowable load-bearing values of Table 1806.2 are used or local conditions on soil capacity prevail, many precautions must be taken in the design of foundation systems that are not regulatory functions of the code, but rather are the professional considerations and design applications of those who are engaged in foundation engineering. The building official needs to pay particular attention to proper selection of the allowable load-bearing capacity of the soil because it is the source of many foundation failures. The problem arises when the load-bearing material directly under a foundation overlies a stratum (or strata) of weaker material having a smaller allowable load-bearing capacity. The selection of the load-bearing value to be used in the design of the foundation should take into account the load distribution at the weaker stratum (or strata) so that the pressure on the soil does not exceed its allowable load-bearing capacity. In this respect, it is important to have a soil profile showing the classes of material at the construction site or at properties in the near vicinity of or adjacent to the area of construction. Such information can be a part of the records or other data required by Section 1803.6 or the results of a soil investigation.

With this information, the building official can consult with the registered design professional in charge of the foundation design to ascertain that due care was exercised in adopting proper soil load-bearing values.

1806.3 Lateral load resistance. Where the presumptive values of Table 1806.2 are used to determine resistance to lateral loads, the calculations shall be in accordance with Sections 1806.3.1 through 1806.3.4.

- When the tabulated values for lateral load resistance of soils are utilized, certain limitations must be considered, and they are contained in the following subsections.

1806.3.1 Combined resistance. The total resistance to lateral loads shall be permitted to be determined by combining the values derived from the lateral bearing pressure and the lateral sliding resistance specified in Table 1806.2.

- This section permits a foundation's lateral resistance to be determined by combining the lateral bearing and lateral sliding values from Table 1806.2.

1806.3.2 Lateral sliding resistance limit. For clay, sandy clay, silty clay, clayey silt, silt and sandy silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

- For cohesive soils, the tabulated lateral sliding value is set at 130 psf (6.23 kPa). However, this section also stipulates that for clay, sandy clay, silty clay and clayey silt, the lateral sliding resistance cannot exceed one-half the dead load.

<table>
<thead>
<tr>
<th>CLASS OF MATERIALS</th>
<th>VERTICAL FOUNDATION PRESSURE (psf)</th>
<th>LATERAL BEARING PRESSURE (psf/ft below natural grade)</th>
<th>LATERAL SLIDING RESISTANCE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12,000</td>
<td>1,200</td>
<td>0.70</td>
</tr>
<tr>
<td>1. Crystalline bedrock</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Sedimentary and foliated rock</td>
<td>4,000</td>
<td>400</td>
<td>0.35</td>
</tr>
<tr>
<td>3. Sandy gravel and/or gravel (GW and GP)</td>
<td>3,000</td>
<td>200</td>
<td>0.35</td>
</tr>
<tr>
<td>4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)</td>
<td>2,000</td>
<td>150</td>
<td>0.25</td>
</tr>
<tr>
<td>5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)</td>
<td>1,500</td>
<td>100</td>
<td>130</td>
</tr>
</tbody>
</table>

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.

b. Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2.
1806.3.3 Increase for depth. The lateral bearing pressures specified in Table 1806.2 shall be permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

- The lateral sliding resistance is calculated as the sum of the lateral bearing and lateral sliding values from Table 1806.2. The lateral bearing values are determined as the product of the tabular value and the depth below natural grade. This section essentially limits the foundation depth for which the lateral bearing values can be increased to 15 feet (4572 mm).

1806.3.4 Increase for poles. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 1/2 inch (12.7 mm) motion at the ground surface due to short-term lateral loads shall be permitted to be designed using lateral bearing pressures equal to two times the tabular values.

- For isolated poles used as supports for flagpoles or signs, and poles used to support buildings that are not adversely affected by a 1/2-inch (12.7 mm) motion at the ground surface due to short-term lateral loads, the tabulated lateral bearing values are permitted to be doubled.

SECTION 1807
FOUNDATION WALLS, RETAINING WALLS AND EMBEDDED POSTS AND POLES

1807.1 Foundation walls. Foundation walls shall be designed and constructed in accordance with Sections 1807.1.1 through 1807.1.6. Foundation walls shall be supported by foundations designed in accordance with Section 1808.

- Foundation walls typically serve as the enclosure for a basement or crawl space as well as a below-grade load-bearing foundation component. Where applicable, they are designed to resist lateral soil pressures as well as dead, live, snow, wind, and seismic loads. This section contains the provisions applicable to the design and construction of foundation walls.

1807.1.1 Design lateral soil loads. Foundation walls shall be designed for the lateral soil loads set forth in Section 1610.

- This section requires that foundation and retaining walls be designed to resist the lateral soil loads in accordance with Section 1610. The code provides lateral soil loads in Table 1610.1 for use where a specific geotechnical investigation has not been performed. Consideration must be also be given to hydrostatic loads in addition to lateral pressures resulting from surcharge loads, such as a sloping backfill.

1807.1.2 Unbalanced backfill height. Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height shall be permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.

- This provision establishes unbalanced backfill height as the difference in height of the exterior and interior finished ground levels. The height of unbalanced backfill quantifies the magnitude of the lateral soil load for the different classifications of soils presented in the prescriptive foundation wall tables. For an illustration of unbalanced backfill height where an interior slab on grade is in contact with the foundation wall, see Figure 1807.1.2.

![Figure 1807.1.2 UNBALANCED BACKFILL HEIGHT](image)

1807.1.3 Rubble stone foundation walls. Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundation walls of structures assigned to Seismic Design Category C, D, E or F.

- Foundation walls of rough or random rubble stone are limited to a thickness of 16 inches (406 mm). Because of the use of rough stones of irregular size and shape, a larger thickness is required (as compared to concrete or masonry walls) for adequate bonding of the stone and mortar to provide structural stability against soil pressures, as well as protection from water penetration.

1807.1.4 Permanent wood foundation systems. Permanent wood foundation systems shall be designed and installed in accordance with AF&PA PWF. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.

- This section covers the design and installation of a multicomponent wood foundation system complying with the specifications of AF&PA PWF. The construction is essentially a below-grade, load-bearing, wood-frame system that serves as an enclosure for basements and crawl spaces as well as the structural foundation for the support of light-frame structures. According to AF&PA PWF, such systems are "engineered to support lateral soil pressures as well as dead, live, snow, wind, and seismic loads," where the foundation wall behavior is primary, and vertical load behavior is secondary.
This foundation system consists of several principal components:

1. Walls that consist of plywood panels attached to a wood stud framing system;

2. A composite footing consisting of a continuous wood plate set on a bed of granular materials, such as sand, gravel or crushed stone, which supports the foundation walls and transmits their loads to the bearing soil below;

3. Polyethylene film that serves as a vapor barrier and covers the exterior side of the plywood foundation walls from grade down to the footing plate;

4. Caulking compounds used for sealing joints in plywood walls as well as bonding agents for attaching the polyethylene film to the plywood and for sealing the film joints;

5. Metal fasteners made of silicon, bronze, copper or stainless steel or hot-dipped, zinc-coated steel nails or staples; and

6. Pressure-treated plywood and lumber to protect the foundation material against decay, termites and other insects.

A typical cross section of a wood foundation basement wall is shown in Figure 1807.1.4.

All plywood and lumber used in the foundation system are required to receive a preservative treatment to protect the material against decay, termites and other insects in accordance with American Wood Preserver’s Association (AWPA U1), Specification A, Use Category 4B and Section 5.2. Specification A of AWPA U1 pertains to sawn wood products, while Use Category 4B refers to wood that is in contact with the ground. The additional reference to Section 5.2 incorporates requirements under Specification A that are unique to permanent wood foundations. This section also requires the treated wood to be identified in accordance with Section 2303.1.8.1. See the commentary to that section for a discussion of identification requirements.

Ordinary metal fasteners in contact with preservatives used in the treatment of wood products will corrode over time and can cause serious structural problems. For this reason, AF&PA PWF prescribes that only fasteners made of silicon bronze, copper or Type 304 or 316 stainless steel, as defined by the American Iron and Steel Institute (AISI), are permitted to be used in a wood foundation system. Hot-dipped, zinc-coated steel nails are permitted to be used in a
wood foundation system when installed in accordance with the specific conditions contained in AF&PA PWF for the surface treatment of nails and moisture protection of the foundation.

Certain cold-weather precautions should be taken when installing a wood foundation system. The composite footing consisting of a wood plate supported on a bed of stone or sand fill should not be placed on frozen ground. While the bottom of the wood plate footing would normally be placed below the frost line (i.e., basement construction), under certain drainage conditions, AF&PA PWF permits the wood plate to be set above the frost line level. The building official should ascertain that such an alternative design satisfies the intent of the code. Most important is the use of proper sealants during very cold weather. All manufacturers of sealants and bonding agents impose temperature restrictions on the use of their products. Only sealants and bonding agents specifically produced for cold-weather conditions should be used. This is an important consideration since sealants are a factor in verifying the water-tight integrity of the foundation installation.

The wood foundation system is an innovative design consisting of many parts and several kinds of materials. It can be expected to perform satisfactorily as a foundation for light construction only if the comprehensive provisions and recommendations of AF&PA PWF are strictly followed.

1807.1.5 Concrete and masonry foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21, as applicable.

Exception: Concrete and masonry foundation walls shall be permitted to be designed and constructed in accordance with Section 1807.1.6.

- Foundation walls are usually designed and constructed in accordance with the accepted engineering practices to carry the vertical loads from the structure above, resist wind and any lateral forces transmitted to the foundations and sustain earth pressures exerted against the walls, including any forces that may be imposed by frost action. For reinforced and plain concrete, the physical properties and design criteria are provided in Chapter 19. For masonry construction, the physical properties and design criteria are provided in Chapter 21. An exception permits the use of prescriptive foundation walls when the applicable limitations are met.

1807.1.6 Prescriptive design of concrete and masonry foundation walls. Concrete and masonry foundation walls that are laterally supported at the top and bottom shall be permitted to be designed and constructed in accordance with this section.

- This section allows prescriptive design of concrete or masonry foundation walls that are primarily intended for, but not necessarily limited to, basement construction in residential and light commercial buildings or other light structures. For the actual load limit on these prescriptive concrete and masonry foundation walls, see Section 1807.1.6.2, Item 7 and Section 1807.1.6.3, Item 8, respectively. Since the designs in the tables are only applicable to foundation walls that are laterally supported at the top and bottom, foundation walls that are not laterally supported at the top and bottom must be designed as described in Section 1807.1.5.

- 1807.1.6.1 Foundation wall thickness. The thickness of prescriptively designed foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8-inch (203 mm) nominal width shall be permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 1807.1.6.2 or 1807.1.6.3 are met.

- The minimum thicknesses in this section are to facilitate the support of the wall above grade. The thickness requirements in this section are empirical and have been used successfully for many years.

- 1807.1.6.2 Concrete foundation walls. Concrete foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.2.

2. The size and spacing of vertical reinforcement shown in Table 1807.1.6.2 is based on the use of reinforcement with a minimum yield strength of 60,000 pounds per square inch (psi) (414 MPa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 MPa) or 50,000 psi (345 MPa) shall be permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.97 or 0.83, respectively.

3. Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, d, from the outside face (soil face) of the wall. The distance, d, is equal to the wall thickness, t, minus 1.25 inches (32 mm) plus one-half the bar diameter, d_μ (\( d = t - (1.25 + d_μ / 2) \)). The reinforcement shall be placed within a tolerance of ± 1/8 inch (9.5 mm) where d is less than or equal to 8 inches (203 mm) or ± 1/4 inch (12.7 mm) where d is greater than 8 inches (203 mm).

4. In lieu of the reinforcement shown in Table 1807.1.6.2, smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length shall be permitted.

5. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 5/8 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1 inch (25 mm) for No. 5 bars and smaller, and not less than 2 inches (51 mm) for larger bars.

6. Concrete shall have a specified compressive strength, \( f'_c \), of not less than 2,500 psi (17.2 MPa).
7. The unfactored axial load per linear foot of wall shall not exceed 1.2 t f', where t is the specified wall thickness in inches.

- The section contains the limitations on construction details and material properties that are needed to utilize the prescriptive designs of Table 1807.1.6.2 for concrete foundation walls. The concrete wall thickness and reinforcing (if any) must be as shown in Table 1807.1.6.2, based on the wall's height and depth of any supported backfill as well as the applicable lateral soil pressure.

- The minimum required concrete strength is given as well as the yield strength of reinforcement steel that was used as the basis for the tabulated vertical reinforcement. The use of other grades of reinforcing steel is permitted provided the appropriate adjustment is made to the spacing of the vertical bars.

**TABLE 1807.1.6.2.** See below.

- The wall thickness and, where required, the size and spacing of reinforcing bars in the table are based on the design requirements from ACI 318.

Design lateral soil pressures correspond to those for some, but not all, of the soils that are tabulated in Table 1610.1. Section 1610.1 requires that the Table 1610.1 lateral pressures be used unless other values are substantiated by a geotechnical investigation. Note d coordinates with requirements in Section 1610.1. Generally, foundation walls that are restrained at the top must be designed for at-rest lateral soil loads. An exception permits the use of active pressures for designing walls that are no more than 8 feet (2438 mm) in height and are supported at the top by a flexible diaphragm. Note that in Table 1610.1, the minimum at-rest design lateral load is 60 psf per foot (2873 Pa/mm) of depth. Depending upon the soil's classification (see Table 1610.1), the prescriptive foundation wall table does not apply to soils with design lateral pressures exceeding 60 psf per foot (2873 Pa/mm) of depth.

Footnote c explains that "PC" as used in this table means "plain concrete," but it actually indicates that

<table>
<thead>
<tr>
<th>MAXIMUM WALL HEIGHT (feet)</th>
<th>MAXIMUM UNBALANCED BACKFILL HEIGHT* (feet)</th>
<th>MINIMUM VERTICAL REINFORCEMENT-BAR SIZE AND SPACING (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design lateral soil load (psf per foot of depth)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Minimum wall thickness (inches)</td>
</tr>
<tr>
<td>7.5</td>
<td>9.5</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>4 PC</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.

b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.2.

c. "PC" means plain concrete.

d. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable (see Section 1610).

e. For height of unbalanced backfill, see Section 1807.1.2.
no vertical reinforcing is required. The term is not to be misconstrued with the more general use of the term “plain concrete” in ACI 318, Chapter 19, as well as Section 1807.1.6.2.1.

1807.1.6.2.1 Seismic requirements. Based on the seismic design category assigned to the structure in accordance with Section 1613, concrete foundation walls designed using Table 1807.1.6.2 shall be subject to the following limitations:

1. Seismic Design Categories A and B. Not less than one No. 5 bar shall be provided around window, door and similar sized openings. The bar shall be anchored to develop $f_y$ in tension at the corners of openings.

2. Seismic Design Categories C, D, E and F. Tables shall not be used except as allowed for plain concrete members in Section 1905.1.8.

- For structures classified as either Seismic Design Category A or B, the prescriptive concrete foundation wall Table 1807.1.6.2 applies along with the provisions stated in Item 1 of this section. For other seismic design categories, the use of the prescriptive concrete foundation wall Table 1807.1.6.2 is limited by Section 1905.1.8. That section allows the use of plain concrete foundation walls (i.e., walls that do not comply with the ACI 318 definition of “reinforced concrete”) in one- and two-family dwellings.

1807.1.6.3 Masonry foundation walls. Masonry foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.3(1) for plain masonry walls or Table 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4) for masonry walls with reinforcement.

2. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 MPa).

3. The specified location of the reinforcement shall equal or exceed the effective depth distance, $d$, noted in Tables 1807.1.6.3(2), 1807.1.6.3(3) and 1807.1.6.3(4) and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in TMS 602/ACI 530.1/ASCE 6, Article 3.4.B.8 of the specified location.

4. Grout shall comply with Section 2103.13.

5. Concrete masonry units shall comply with ASTM C 90.

6. Clay masonry units shall comply with ASTM C 652 for hollow brick, except compliance with ASTM C 62 or ASTM C 216 shall be permitted where solid masonry units are installed in accordance with Table 1807.1.6.3(1) for plain masonry.

7. Masonry units shall be laid in running bond and installed with Type M or S mortar in accordance with Section 2103.9.

8. The unfactored axial load per linear foot of wall shall not exceed $1.2 \cdot f_y$ where $f_y$ is the specified wall thickness in inches and $f_y$ is the specified compressive strength of masonry in pounds per square inch.

9. At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.

10. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbelled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm). The hollow space behind the corbelled masonry shall be filled with mortar or grout.

- This section contains the limitations on construction details and material properties that are needed to utilize the prescriptive designs of Tables 1807.1.6.3(1) through 1807.1.6.3(4) for masonry foundation walls.

- Item 3 provides specific placement requirements regarding the location of reinforcement in connection with Tables 1807.1.6.3(2) through 1807.1.6.3(4). This dimension is necessary so that the wall is capable of developing the required bending capacity to resist the lateral soil loads.

- Where corbeling is utilized to match the width of a masonry cavity wall above, Item 10 directs the reader to the appropriate requirements in Chapter 21. A header course is provided at the corbel to transfer the load deeper into the supporting wall. An alternative to the header utilizes a filled collar joint and ties to accomplish this load transfer. This option is illustrated in Figure 1807.1.6.3.

TABLE 1807.1.6.3(1). See page 18-26.

- This table provides wall thickness designs for various wall heights and unbalanced backfill depths. The thicknesses in the table are proportional to the lateral soil loads for each classification of soils. The lateral soil loads in the table are consistent with some of the soil loads in Table 1610.1.

- The thicknesses in the table for plain masonry are based on research by the National Concrete Masonry Association (NCMA) in the report Research Evaluation of the Flexural Strength of Concrete Masonry, Project No. 93-172, December 7, 1993.

- Footnote f provides a caution related to the applicability of 30 and 45 psf (1.43 and 2.156 kPa) design lateral soil loads. Section 1610.1 requires that foundation walls be designed for at-rest pressures, except foundation walls up to 8 feet (2438 mm) in height that are supported at the top by a flexible diaphragm. Where lateral soil loads from Table 1610.1 are used, the values of 30 and 45 psf (1.43 and 2.156 kPa) occur only for active pressure. Therefore, where both of these conditions apply, the prescriptive design table entries for 30 and 45 psf (1.43 and 2.156 kPa) cannot be used.
SOILS AND FOUNDATIONS

For SI: 1 inch = 25.4 mm.

**Figure 1807.1.6.3**
TIE AT TOP COURSE BED JOINT

**TABLE 1807.1.6.3(1)**
PLAIN MASONRY FOUNDATION WALLS

<table>
<thead>
<tr>
<th>MAXIMUM WALL HEIGHT (feet)</th>
<th>MAXIMUM UNBALANCED BACKFILL HEIGHT* (feet)</th>
<th>MINIMUM NOMINAL WALL THICKNESS (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>30'</td>
</tr>
<tr>
<td>7</td>
<td>4 (or less)</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>12</td>
</tr>
<tr>
<td>8</td>
<td>4 (or less)</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>10 (solid')</td>
</tr>
<tr>
<td>9</td>
<td>4 (or less)</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>12 (solid')</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>12 (solid')</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.
c. Solid grouted hollow units or solid masonry units.
d. A design in compliance with Chapter 21 or reinforcement in accordance with Table 1807.1.6.3(2) is required.
e. For height of unbalanced backfill, see Section 1807.1.2.
f. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable (see Section 1610).
g. For height of unbalanced backfill, see Section 1807.1.2.
h. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable (see Section 1610).
TABLE 1807.1.6.3(2). See below.

For general comments regarding this table, see the commentary to Table 1807.1.6.3(1). The reinforcement size and spacing in the table are based on the design requirements from TMS 402/ACI 530/ASCE 5. Three different tables are provided for three distinct values of effective depth, d. For an explanation of the effective depth, see the commentary to Section 1807.1.6.3.

TABLE 1807.1.6.3(3). See page 18-28.

See the commentary to Table 1807.1.6.3(2).

TABLE 1807.1.6.3(4). See page 18-29.

See the commentary to Table 1807.1.6.3(2).

1807.1.6.3.1 Alternative foundation wall reinforcement. In lieu of the reinforcement provisions for masonry foundation walls in Table 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4), alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall shall be permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.

The purpose of this section is to provide an alternative for the reinforcement requirements in Tables 1807.1.6.3(2), 1807.1.6.3(3) and 1807.1.6.3(4) that would result in the same capacity for lateral soil load resistance. For example, if No. 5 reinforcement bars were installed where the table indicates No. 4 at 48 inches (1219 mm) on center (o.c.), the spacing of the No. 5 bars for equivalent reinforcement area per foot of wall would be 72 inches (1829 mm) o.c. The increase in spacing would be allowed, since the cross-sectional area of a No. 5 bar is approximately 50 percent higher than a No. 4 reinforcement bar.

1807.1.6.3.2 Seismic requirements. Based on the seismic design category assigned to the structure in accordance with Section 1613, masonry foundation walls designed using Tables 1807.1.6.3(1) through 1807.1.6.3(4) shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements.

<table>
<thead>
<tr>
<th>MAXIMUM WALL HEIGHT (feet-inches)</th>
<th>MAXIMUM UNBALANCED BACKFILL HEIGHT* (feet-inches)</th>
<th>MINIMUM VERTICAL REINFORCEMENT-BAR SIZE AND SPACING (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design lateral soil load (psf per foot of depth)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30°</td>
</tr>
<tr>
<td>7-4</td>
<td>4-0 (or less)</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>7-4</td>
<td>#5 at 48</td>
</tr>
<tr>
<td>8-0</td>
<td>4-0 (or less)</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td>8-8</td>
<td>4-0 (or less)</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td>9-4</td>
<td>4-0 (or less)</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>9-4*</td>
<td>#7 at 48</td>
</tr>
<tr>
<td>10-0</td>
<td>4-0 (or less)</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 48</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 48</td>
</tr>
<tr>
<td></td>
<td>9-0*</td>
<td>#7 at 48</td>
</tr>
<tr>
<td></td>
<td>10-0*</td>
<td>#7 at 48</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610.

b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.

c. For alternative reinforcement, see Section 1807.1.6.3.1

d. For height of unbalanced backfill, see Section 1807.1.2

e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.
2. **Seismic Design Category C**. A design using Tables 1807.1.6.3(1) through 1807.1.6.3(4) is subject to the seismic requirements of Section 1.18.4.3 of TMS 402/ACI 530/ASCE 5.

3. **Seismic Design Category D**. A design using Tables 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 1.18.4.4 of TMS 402/ACI 530/ASCE 5.

4. **Seismic Design Categories E** and **F**. A design using Tables 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 1.18.4.5 of TMS 402/ACI 530/ASCE 5.

For structures classified as either Seismic Design Category A or B, the prescriptive masonry foundation wall tables apply without further requirements. For all other seismic design categories, the use of these prescriptive tables is limited by the referenced code sections. For structures classified as Seismic Design Categories D or higher, only the tables that require reinforcement can be used.

### TABLE 1807.1.6.3(3)

**10-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 6.75 INCHES**

<table>
<thead>
<tr>
<th>MAXIMUM WALL HEIGHT (feet-inches)</th>
<th>MAXIMUM UNBALANCED BACKFILL HEIGHT* (feet-inches)</th>
<th>MINIMUM VERTICAL REINFORCEMENT-BAR SIZE AND SPACING (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design lateral soil load* (psf per foot of depth)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30°</td>
</tr>
<tr>
<td>7-4</td>
<td>4-0 (or less)</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>7-4</td>
<td>#4 at 56</td>
</tr>
<tr>
<td>8-0</td>
<td>4-0 (or less)</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 56</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 56</td>
</tr>
<tr>
<td>8-8</td>
<td>4-0 (or less)</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 56</td>
</tr>
<tr>
<td></td>
<td>8-8</td>
<td>#5 at 56</td>
</tr>
<tr>
<td>9-4</td>
<td>4-0 (or less)</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 56</td>
</tr>
<tr>
<td></td>
<td>8-8</td>
<td>#6 at 56</td>
</tr>
<tr>
<td></td>
<td>9-4</td>
<td>#6 at 56</td>
</tr>
<tr>
<td>10-0</td>
<td>4-0 (or less)</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 56</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#5 at 56</td>
</tr>
<tr>
<td></td>
<td>8-8</td>
<td>#6 at 56</td>
</tr>
<tr>
<td></td>
<td>9-0</td>
<td>#7 at 56</td>
</tr>
<tr>
<td></td>
<td>10-8</td>
<td>#7 at 56</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8, 1 pound per square foot per foot = 1.157 kPa/m.

a. For design lateral soil loads, see Section 1610.

b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.

c. For alternative reinforcement, see Section 1807.1.6.3.1.

d. For height of unbalanced backfill, see Section 1807.1.2.

e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

1807.2 Retaining walls. Retaining walls shall be designed in accordance with Sections 1807.2.1 through 1807.2.3.

- This section provides design considerations for a retaining wall.

1807.2.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, lateral soil pressures on both sides of the keyway shall be considered in the sliding analysis.

- This provision for retaining wall analyses is intended to provide guidance on the considerations that are appropriate for retaining wall design. Designs for retaining walls with keyways should account for the driving pressures acting on the keyway. Since there are indications that these are at times overlooked, this provision clarifies that such forces must be considered.
1807.2.2 Design lateral soil loads. Retaining walls shall be designed for the lateral soil loads set forth in Section 1610.

- Section 1610.1 requires that the Table 1610.1 lateral pressures be used unless other values are substantiated by a geotechnical investigation. The lateral pressure of the soil against the retaining wall is greatly influenced by soil moisture. Backfill is usually kept from being saturated for an extended length of time by placing drains near the base of the retaining wall to remove the water in the soil behind it.

1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a minimum safety factor of 1.5 in each case. The load combinations of Section 1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 1.0 times other nominal loads, and investigation with one or more of the variable loads set to zero. The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the retaining wall divided by the net lateral force applied to the retaining wall.

Exception: Where earthquake loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

- The safety factor for stability of retaining walls predates the development of load combinations. This section clarifies that load combinations do not apply when evaluating the stability of retaining walls under sliding and overturning.

1807.3 Embedded posts and poles. Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or in concrete footings in earth shall be in accordance with Sections 1807.3.1 through 1807.3.3.

- The criteria in this section apply to the lateral resistance of posts or poles embedded either directly in earth or in a concrete footing. They were originally developed to reduce the complexity of analyzing the

---

### TABLE 1807.1.6.3(4)

<table>
<thead>
<tr>
<th>12-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE ( d \geq 8.75 ) INCHES(^a, b, c)</th>
<th>MINIMUM VERTICAL REINFORCEMENT-BAR SIZE AND SPACING (inches)</th>
<th>30(^b)</th>
<th>45(^b)</th>
<th>60(^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAXIMUM WALL HEIGHT (feet-inches)</td>
<td>MAXIMUM UNBALANCED BACKFILL HEIGHT(^d) (feet-inches)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-4</td>
<td>4 (or less)</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
</tr>
<tr>
<td></td>
<td>7-4</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
</tr>
<tr>
<td>8-0</td>
<td>4 (or less)</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
<td>#8 at 72</td>
</tr>
<tr>
<td>8-8</td>
<td>4 (or less)</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
<td>#7 at 72</td>
</tr>
<tr>
<td></td>
<td>8-8(^e)</td>
<td>#5 at 72</td>
<td>#7 at 72</td>
<td>#8 at 72</td>
</tr>
<tr>
<td>9-4</td>
<td>4 (or less)</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#5 at 72</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
<td>#7 at 72</td>
</tr>
<tr>
<td></td>
<td>8-8(^e)</td>
<td>#6 at 72</td>
<td>#7 at 72</td>
<td>#8 at 72</td>
</tr>
<tr>
<td>10-0</td>
<td>4 (or less)</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>5-0</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
<td>#4 at 72</td>
</tr>
<tr>
<td></td>
<td>6-0</td>
<td>#4 at 72</td>
<td>#5 at 72</td>
<td>#5 at 72</td>
</tr>
<tr>
<td></td>
<td>7-0</td>
<td>#4 at 72</td>
<td>#6 at 72</td>
<td>#6 at 72</td>
</tr>
<tr>
<td></td>
<td>8-0</td>
<td>#5 at 72</td>
<td>#6 at 72</td>
<td>#7 at 72</td>
</tr>
<tr>
<td></td>
<td>9-0(^e)</td>
<td>#6 at 72</td>
<td>#7 at 72</td>
<td>#8 at 72</td>
</tr>
<tr>
<td></td>
<td>10-0(^e)</td>
<td>#7 at 72</td>
<td>#8 at 72</td>
<td>#9 at 72</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/in.

- a. For design lateral soil loads, see Section 1610.
- b. Provisions for this table are based on design and construction requirements specified in Section 1807.1.6.3.
- c. For alternative reinforcement, see Section 1807.1.6.3.1.
- d. For height of unbalanced backfill, see Section 1807.1.2.
- e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.
SOILS AND FOUNDATIONS

soil-structure interaction, thereby facilitating the design of cantilevered poles used to support outdoor advertising. While more accurate methods are available for determining the resistance of posts or poles to axial and lateral loads, the other methods are much more complex, and the results do not differ significantly from this method.

**1807.3.1 Limitations.** The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.

2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

Wood poles shall be treated in accordance with AWPA U1 for sawn timber posts (Commodity Specification A, Use Category 4B) and for round timber posts (Commodity Specification B, Use Category 4B).

- The limitations imposed by this section are intended to address both structural stability and serviceability. The limitation of the frictional resistance for structural walls and slabs on silts and clays is consistent with Section 1806.3 and Table 1806.2, which also limit the sliding resistance to one-half the dead load. The limitations on the types of construction materials that utilize lateral support of poles are based on the brittle nature of the materials. In order to prevent excessive distortions, which would produce cracking of these brittle materials, this section limits the use of poles unless some type of rigid cross bracing is provided to limit the deflections to those that can be tolerated by the materials.

**1807.3.2 Design criteria.** The depth to resist lateral loads shall be determined using the design criteria established in Sections 1807.3.2.1 through 1807.3.2.3, or by other methods approved by the building official.

- The design criteria for the use of poles embedded in the ground or in concrete footings in the ground were originally developed in the 1940s. The design criteria address conditions where constraint is provided at the ground surface, such as a rigid floor, and where no constraint is provided. The original design criteria established a \( \frac{1}{4} \) inch (12.7 mm) lateral pole deformation at the surface of the ground. [Note that where \( \frac{1}{4} \) inch (12.7 mm) movement can be tolerated, Section 1806.3.4 allows the lateral bearing pressure to be increased.] These criteria were also based on field tests conducted in a range of sandy and gravelly soils and of silts and clays.

**1807.3.2.1 Nonconstrained.** The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

\[
d = 0.5A\{1 + [1 + (4.3b/A)]^{1/2}\} \quad (\text{Equation 18-1})
\]

where:

\[
A = 2.34P/S_1b
\]

\( b \) = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

\( d \) = Depth of embedment in earth in feet (m) but not over 12 feet (3.658 m) for purpose of computing lateral pressure.

\( h \) = Distance in feet (m) from ground surface to point of application of "\( P \)."

\( P \) = Applied lateral force in pounds (kN).

\( S_1 \) = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

- The required embedment depth for posts or poles that are not restrained at or above the ground surface is determined by Equation 18-1. Nonconstrained conditions include flexible pavements, such as asphalt. The code further restricts the use of this equation to structures that have no lateral support, such as from a floor diaphragm, provided above the ground level. From an analysis standpoint, the embedded pole is free to rotate and translate at the point of load application. The pole rotates about a point below the soil surface, which is where the direction of the passive pressure on the embedded pole changes direction.

Determining the minimum embedment depth of a vertical member resisting lateral loads requires two steps:

1. Assuming an embedment depth, calculate the soil pressure that the embedded member imposes on the soil, and

2. Compare the calculated pressure to an allowable lateral soil bearing pressure.

This two-step process is expressed as a single trial-and-error equation, which requires an iterative process to arrive at a satisfactory depth of embedment.

Note that the allowable lateral soil bearing pressure is in accordance with Section 1806.2. This includes increases permitted in Section 1806.3.4. In particular, the allowable lateral bearing values can be doubled for poles supporting structures that can tolerate 0.5 inches (12.7 mm) of movement at the ground surface—generally considered applicable to isolated poles.

**1807.3.2.2 Constrained.** The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

\[
d = \frac{A_{2.25}Pb}{S_1} \quad (\text{Equation 18-2})
\]
or alternatively

\[ d = \frac{4.25 M_g}{S_b b} \]  

(Equation 18-2)

where:

- \( M_g \) = Moment in the post at grade, in foot-pounds (kN·m).
- \( S_b \) = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

The required embedment depth for posts or poles that are restrained at the ground surface is determined by Equation 18-2 or 18-3. A constrained post has stiff resistance at grade, compared to the stiffness of the soil. This point of support acts as a fulcrum for the lateral load. The pole is free to rotate about this point rather than being fixed.

**1807.3.2.3 Vertical load.** The resistance to vertical loads shall be determined using the vertical foundation pressure set forth in Table 1806.2.

While no specific limit is provided with respect to vertical loading, this methodology was originally developed for cantilevered posts where the primary concern is lateral resistance (see commentary, Section 1807.3). The above determination of embedment length is independent of the vertical load (see the commentary, Section 1806.2 and Table 1806.2).

**1807.3.3 Backfill.** The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with a specified compressive strength of not less than 2,000 psi (13.8 MPa). The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

In order for the post or pole to meet the conditions and limitations of research for which the above criteria was established, backfill in the annular space around a column not embedded in a concrete footing must be either 2,000 psi (13.8 MPa) concrete, CLSM (see commentary, Section 1804.6) or clean sand thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.

**SECTION 1808 FOUNDATIONS**

**1808.1 General.** Foundations shall be designed and constructed in accordance with Sections 1808.2 through 1808.9. Shallow foundations shall also satisfy the requirements of Section 1809. Deep foundations shall also satisfy the requirements of Section 1810.

- Section 1808 contains the provisions that apply to all foundation types. Then specific requirements for shallow and deep foundations are given in Sections 1809 and 1810, respectively.

**1808.2 Design for capacity and settlement.** Foundations shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. Foundations in areas with expansive soils shall be designed in accordance with the provisions of Section 1808.6.

Regardless of the type of shallow foundation used, the allowable bearing capacity of soil must not be exceeded. There are two premises by which allowable soil-bearing pressures are established. The first premise requires that the safety factor against ultimate shear failure of the soil be adequate. The second premise requires that settlements under allowable bearing pressures not exceed tolerable values. In most cases, settlement governs the value established for allowable soil-bearing capacity. Bearing capacity is usually determined from a soils investigation and engineering analysis.

When the soils profile of a construction site is established by a sufficient number of test borings, and it indicates that a nonuniform soil condition exists where the strata of suitable bearing materials occurs at varying thicknesses or different depths, the foundation design must be adjusted to the subsurface condition to provide for the proper and safe performance of the foundation system.

Under such circumstances, it becomes necessary in the design of shallow foundations to determine the different depths at which isolated or continuous stepped footings need to be placed in order to obtain equal bearing pressures and avoid serious structural damage caused by differential (unequal) settlement of the different parts of the foundation system.

Another design method for obtaining equal bearing pressures and keeping the footings at a common elevation is to size the footings in accordance with the allowable bearing capacity of the soil at each location, thus producing a balanced design of the foundation system and preventing differential settlement.

Section 1808.6 stipulates required methods and design of foundations on expansive soils.

**1808.3 Design loads.** Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605.2 or 1605.3. The dead load is permitted to include the weight of foundations and overlying fill. Reduced live loads, as specified in Sections 1607.10 and 1607.12, shall be permitted to be used in the design of foundations.

Foundations are permitted to be designed using either ASD or strength design. The appropriate load combinations are investigated to determine the most severe structural effects. Live load reductions permit-
SOILS AND FOUNDATIONS

ted in Section 1607 apply equally to the foundation design loads. This section also clarifies what portions of the foundation construction are considered dead loads. See the commentary to Section 1606 for a discussion of dead load estimates.

1808.3.1 Seismic overturning. Where foundations are proportioned using the load combinations of Section 1605.2 or 1605.3.1, and the computation of seismic overturning effects is by equivalent lateral force analysis or modal analysis, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.

- This provision correlates with ASCE 7 earthquake load requirements that are based on the NEHRP Recommended Provisions for Seismic Regulations for New Buildings (FEMA 450). When using LRFD load combinations to size foundations, the seismic overturning computed by the equivalent lateral force method or the modal analysis method is permitted to be reduced.

ASCE 7 permits the reduction of seismic overturning for foundation design where either strength design or ASD load combinations are used. This provision refers to Sections 1605.2 and 1605.3.1, which correspond to the ASCE 7 load combinations. Because the load combinations in Section 1605.3.2 include 0.9D, where overturning is assessed (rather than 0.6D as in Section 1605.3.1), reduction of seismic overturning would be unconservative where those load combinations are used.

1808.4 Vibratory loads. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the foundation design to prevent detrimental disturbances of the soil.

- Vibrations emanating from machinery operations and transmitted to the soil through the foundation may cause serious settlement to occur, particularly to foundations bearing on granular materials. While granular soils generally have a considerable volume of voids, the foundation pressure is usually carried and distributed by the bearing of grain on grain without detrimental deformations. Granular materials subjected to strong vibrations, however, may result in the particles readjusting and slipping into the void spaces. Essentially, the soil mass is consolidated and reduced in volume, causing vertical settlement.

Vibratory loads that cause a disturbance of the soil will flush water out of the material. In saturated granular soils, such as sand, the flushing action may cause a "quick" condition that allows sudden flow beneath the foundation, sometimes resulting in serious structural damage.

In soils that are rather impermeable, such as clays, vibration may also cause water to flush out, but it will take a very long time. Also, foundation settlement will occur over a prolonged period.

Care should be taken in the design of foundations to eliminate completely or minimize the transmission of vibratory loads to load-bearing soils.

1808.5 Shifting or moving soils. Where it is known that the shallow subsoils are of a shifting or moving character, foundations shall be carried to a sufficient depth to ensure stability.

- Shallow foundations placed on or within a mass of shifting or moving soils must be designed with great rigidity and strength in order to adequately resist soil movement and avoid damage. This section requires foundations to be carried to a sufficient depth for adequate stability. Adequate stability implies, among other things, the consideration of uplift and perpendicular forces exerted on the footings because of soil movement.

1808.6 Design for expansive soils. Foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1808.6.1 or 1808.6.2.

Exception: Foundation design need not comply with Section 1808.6.1 or 1808.6.2 where one of the following conditions is satisfied:

1. The soil is removed in accordance with Section 1808.6.3; or

2. The building official approves stabilization of the soil in accordance with Section 1808.6.4.

- Expansive soils, often referred to as "swelling soils," contain montmorillonite minerals and have the characteristics of absorbing water and swelling, or shrinking and cracking when drying. Significant volume changes can cause serious damage to buildings and other structures as well as to pavements and sidewalks. Swelling soils are found throughout the nation, but are more prevalent in regions with dry or moderately arid climates. It is in areas where these soils experience large fluctuations in water content that the swelling is more likely to affect structures. Heavy rains will cause the soil to expand and push upward on a foundation. Movement is most noticeable at the perimeter of the foundation where the increase in water content is greatest.

There is a general relationship between the PI of a soil as determined by the ASTM D 4318 standard test method and the potential for expansion. This relationship is shown in Figure 1803.5.3.

Section 1803.5.3 defines "Expansive soil" as any plastic material with a plasticity index of 15 or greater with more than 10 percent of the soil particles passing a No. 200 sieve and less than 5 micrometers in size. As an alternative, tests in accordance with ASTM D 4829 can be used to determine if a soil is expansive. The expansion index is a measure of the swelling potential of the soil.

The amount and depth of potential swelling that can occur in a clay material are, to some extent, functions of the cyclical moisture content in the soil. In dryer climates where the moisture content in the soil near the ground surface is low because of evaporation, there is a greater potential for extensive swelling than the same soil in wetter climates where the varia-
tions of moisture content are not as severe.

Volume changes in highly expansive soils range
between 7 and 10 percent, but experience has shown
that occasionally, under abnormal conditions, they
can reach as high as 25 percent.

The exception refers to the provisions that govern
removal or stabilization of the expansive soil (see the
commentary to Section 1803.5.3 for a discussion
regarding expansive soils).

1808.6.1 Foundations. Foundations placed on or within
the active zone of expansive soils shall be designed to resist dif-
ferential volume changes and to prevent structural damage to
the supported structure. Deflection and racking of the sup-
ported structure shall be limited to that which will not inter-
fere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or
below expansive soil shall comply with the following provi-
sions:

1. Foundations extending into or penetrating expansive
soils shall be designed to prevent uplift of the supported
structure.

2. Foundations penetrating expansive soils shall be
designed to resist forces exerted on the foundation due
to soil volume changes or shall be isolated from the
expansive soil.

\* Shallow foundation systems placed on or within a
mass of expansive soil must be designed with great
rigidity and strength in order to adequately resist
swelling pressures and avoid serious structural dam-
age. Sometimes footings and piers, as well as foun-
dation walls and grade beams, are isolated from the
swelling soils by intervening fills or granular materi-
als. While this type of insulation serves to cushion or
diminish lateral pressures, it will not prevent structure
heaving, except for grade beams constructed on col-
lapsible forms.

One method of foundation construction in expans-
soil is the use of drilled piers with belled foot-
ings extending below the zone of swelling activity, or
at least to a soil stratum where the seasonal moisture
content of the expansive soil will remain within a tol-
erable range. This type of construction is made possi-
ble in swelling soils because the material usually
consists of stiff clays that do not contain free water,
providing excellent conditions for drilling holes in the
ground.

Piers with belled footings have been widely used
for foundations in expansive soils, even in the con-
struction of single-family dwellings.

The concrete shaft must be reinforced for its entire
length because the swelling soil is apt to exert high
uplift forces and subject the drilled pier construction
to tensile stresses. This condition can be prevented
or sufficiently mitigated by isolating the concrete from
the swelling soil by surrounding the shaft with a ver-
tical layer of granular soil or other suitable materials
that possess little or no shearing strength (e.g., ver-
miculite).

1808.6.2 Slab-on-ground foundations. Moments, shears
and deflections for use in designing slab-on-ground, mat or
raft foundations on expansive soils shall be determined in
accordance with WRI/CRSI Design of Slab-on-Ground Foun-
dations or PTI Standard Requirements for Analysis of Shal-
low Concrete Foundations on Expansive Soils. Using the
moments, shears and deflections determined above, nonpre-
stressed slabs-on-ground, mat or raft foundations on expan-
sive soils shall be designed in accordance with WRI/CRSI
Design of Slab-on-Ground Foundations and post-tensioned
slab-on-ground, mat or raft foundations on expansive soils
shall be designed in accordance with PTI Standard Require-
ments for Design of Shallow Post-Tensioned Concrete Foun-
dations on Expansive Soils. It shall be permitted to analyze
and design such slabs by other methods that account for soil-
structure interaction, the deformed shape of the soil support,
the plate or stiffened plate action of the slab as well as both
center lift and edge lift conditions. Such alternative methods
shall be rational and the basis for all aspects and parameters
of the method shall be available for peer review.

\* It is not uncommon for concrete slabs on ground to
be reinforced (prestressed) with post-tensioned
strands so that the flexural stresses induced in the
slab from uplift of the soil are reduced, thus avoiding
cracking of the concrete at its top surface. All things
being equal, a post-tensioned slab on ground can be
more economical than a conventionally reinforced
slab because the precompression reduces the flex-
ural stresses, making it unnecessary to increase the
thickness of the slab or use a higher strength con-
crete. It has also been a practice to cast the slab on
cellular forms made of cardboard or other collapsible
materials that will support wet concrete, but will yield
when subjected to swelling pressures. Since there is no
rebound of the form material after subsidence of the
soil, floors must be designed as structural slabs.

This section recognizes the analytical procedures
in Wire Reinforcing Institute/Concrete Reinforcing
Steel Institute (WRI/CRSI) Design of Slab-on-Ground
Foundations and Post-tensioning Institute (PTI) Stan-
dard Requirements for Analysis of Shallow Concrete
Foundations on Expansive Soils for slab-on-ground
foundations located on expansive soils. The WRI/
CRSI document also provides design requirements
for reinforced concrete slabs. The PTI Standard
Requirements for Design of Shallow Post-Tensioned
Concrete Foundations on Expansive Soils addresses
design requirements for post-tensioned concrete slabs on expansive soils.

1808.6.3 Removal of expansive soil. Where expansive soil is
removed in lieu of designing foundations in accordance with
Section 1808.6.1 or 1808.6.2, the soil shall be removed to a
depth sufficient to ensure a constant moisture content in the
remaining soil. Fill material shall not contain expansive soils
and shall comply with Section 1804.5 or 1804.6.

Exception: Expansive soil need not be removed to the
depth of constant moisture, provided the confining pres-
SOILS AND FOUNDATIONS

The best way to avoid the problems associated with expansive soils is to remove such soil from the construction site and, where necessary, replace it with suitable compacted fill material. Expansive soil shrinks when the water content decreases and swells when the water content increases. To minimize the chances of swelling and shrinking, the code requires removal to a depth that provides a constant moisture content in the soil that is left in place. The exception to removing to this depth alludes to an alternative method of stabilization that is discussed in the commentary to Section 1808.6.4.

1808.6.4 Stabilization. Where the active zone of expansive soils is stabilized in lieu of designing foundations in accordance with Section 1808.6.1 or 1808.6.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

This section allows expansive soils to be stabilized by either chemical, presaturation or dewatering methods or by other equivalent techniques. These methods, however, have limited use. Chemical stabilization may be effectively accomplished by the use of lime thoroughly mixed with the soil and compacted at approximately the optimum moisture content. The purpose of using lime is to reduce the plasticity of the soil, which will reduce the swelling potential (see commentary, Section 1808.6). Since this method requires a uniform mix of lime and soil, however, it is generally limited to compacted fills. There is a method of pressure injecting lime slurry into heavily fissured clay materials, but the method is not appropriate for all site conditions.

Presaturation by flooding the site is rarely a totally effective way to stabilize expansive soils, since it takes a very long time for the water to penetrate to any great depth. Dewatering of expansive soils that consist generally of dense clays without free water is not an effective way to control moisture content. One good method of stabilization is to place sufficient fill material on the site so that the downward pressures of the fill will, to the extent possible, balance the upward swelling pressures of the soil. The effectiveness of this balancing concept depends on the pressures developed by the expansive soil as a function of estimated volume change and the depth of compacted fill material necessary to counteract these pressures. The application of this stabilization method is usually economically feasible when the soil pressures are low, around 500 psf (23.9 kPa); however, pressures occasionally have reached as high as 20 tons per square foot (1915 kPa).

1808.7 Foundations on or adjacent to slopes. The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-

percent slope) shall comply with Sections 1808.7.1 through 1808.7.5.

The provisions of this section apply to buildings placed on or adjacent to slopes steeper than 1 unit vertical in 3 units horizontal (33.3-percent slope).

1808.7.1 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided in Section 1808.7.5 and Figure 1808.7.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

Code Figure 1808.7.1 depicts criteria for locating buildings adjacent to slopes. The setback is intended to provide protection to the structure not only from shallow slope failures (sometimes referred to as "sloughing") but also from erosion and slope drainage. At ascending slopes, the setback also provides access around the building and helps create a light and open-air environment. Where an existing slope is steeper than 1:1, the toe of the slope is assumed to be at a distance determined by the intersection of a horizontal line at the top of the foundation and a line drawn at a 45-degree (0.79 rad) angle to the horizontal line, and terminating at the top of the slope. This determination of slope height and measurement of the setback is illustrated in Figure 1808.7.1(1).

FIGURE 1808.7.1. See page 18-35.

This figure illustrates required setbacks at slopes. The setback is a function of the height of the slope (see commentary, Sections 1808.7.1 and 1808.7.2).

1808.7.2 Foundation setback from descending slope surface. Foundations on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the foundation without detrimental settlement. Except as provided for in Section 1808.7.5 and Figure 1808.7.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

The provisions of this section restrict the placement of footings at the top of slopes so that vertical and lateral support for the footing is provided. This setback is shown in code Figure 1808.7.1. When the slope is greater than one unit vertical in one unit horizontal (100-percent slope), the setback is measured from a
top of the slope that is established as the intersection of the ground surface and an imaginary plane at 45 degrees (0.79 rad) to the horizontal, projected from the toe of the slope. This requirement is illustrated in Figure 1808.7.2. For most conditions, the required setbacks will provide adequate lateral support for the foundations.

1808.7.3 Pools. The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

This section specifies the required setback distances for pools located near ascending or descending slopes. The minimum setback distance is established as one-half the required building setback from Section 1808.7.2. The pool wall that is within 7 feet (2134 mm) of the top slope is required to be self-supporting without support from the soil, and is intended to provide additional safety measures should localized minor sliding and sloughing occur.

1808.7.4 Foundation elevation. On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

- Figure 1808.7.4 depicts the requirements of this section regarding elevation for exterior foundations with respect to the street, gutter or point of inlet of a drainage device. The elevation of the street or gutter shown is that point at which drainage from the site reaches the street or gutter.

This requirement is intended to protect the building from water encroachment in case of heavy or unprecedent rain and may be modified on the approval of the building official if he or she finds that positive drainage slopes are provided to drain water away from the building and that the drainage pattern is not subject to temporary flooding due to landscaping or

---

**Figure 1808.7.1**

FOUNDATION CLEARANCES FROM SLOPES

**Figure 1808.7.1(1)**

BUILDINGS ADJACENT TO ASCENDING SLOPE EXCEEDING 1 TO 1

For SI: 1 foot = 304.8 mm.

For SI: 1 degree = 0.01745 rad.
other impediments to drainage. This section is related more directly to the provisions for site grading in Section 1804.3 than it is to Section 1808.7 requirements for footings adjacent to slopes.

1808.7.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official shall be permitted to require a geotechnical investigation as set forth in Section 1803.3.10.

- This section provides the building official the authority to approve alternative setbacks and clearances from slopes, provided he or she is satisfied that the intent of this section has been met. The building official has the authority to require a foundation investigation by a registered design professional to show that the intent of the code has been met. This item also specifies the parameters that must be considered by the registered design professional in the investigation.

1808.8 Concrete foundations. The design, materials and construction of concrete foundations shall comply with Sections 1808.8.1 through 1808.8.6 and the provisions of Chapter 19.

Exception: Where concrete footings supporting walls of light-frame construction are designed in accordance with Table 1809.7, a specific design in accordance with Chapter 19 is not required.

- The design, construction and materials used for concrete foundations are to comply with Chapter 19 which, of course, references ACI 318, and also the provisions of this section which are specific to concrete foundations.

The exception applies to concrete footings support-

**Figure 1808.7.2**
BUILDINGS ADJACENT TO DESCENDING SLOPE EXCEEDING 1 TO 1

**Figure 1808.7.4**
FOOTING ELEVATION ON GRADED SITES

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
ing walls of light-frame construction when a specific footing design is not provided. The purpose of Table 1808.7 is to specify footing sizes that can be used to safely support walls of light-frame construction. The table is based on anticipated loads on foundations due to wall, floor and roof systems.

1808.8.1 Concrete or grout strength and mix proportions. Concrete or grout in foundations shall have a specified compressive strength (f'_c) not less than the largest applicable value indicated in Table 1808.8.1.

Where concrete is placed through a funnel hopper at the top of a deep foundation element, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (204 mm). Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture.

- This section of the code establishes a minimum compressive strength for concrete and grout that is used in foundations. The code's intent is that concrete or grout used as structural material for foundation construction will be of sufficient strength and durability to satisfy safety requirements.

This section also requires that the concrete mix for deep foundation elements be designed and proportioned to produce a cohesive workable mix with a consistency (slump) ranging between 4 inches (102 mm) and 8 inches (203 mm) where it is placed using a funnel hopper. This provision is within the bounds required for the proper placement of conventional structural concrete for cased cast-in-place elements. The slump requirements stated here, however, should not be considered so absolute that, with the use of engineering judgement, the consistency of the concrete cannot be adjusted to satisfy prevailing conditions of construction. For example, concrete placement conducted under difficult conditions—such as for elements containing heavy reinforcement, for very long cased elements, for element shells driven on steep batters or for other demanding situations—may require the employment of special concrete mixes using reduced quantities of coarse aggregates with corresponding increases in sand and cement content.

Under such conditions, slumps of 4 inches (102 mm) with a tolerance of plus 2 inches (51 mm) or minus 1 inch (25 mm) are in order.

The consistency of concrete required for uncased cast-in-place concrete elements may be altogether different than the slumps specified and used for cased construction. For example, concrete placed in drilled holes should have a slump of at least 6 inches (152 mm) so that the concrete flows properly and full bond is attained with the reinforcement.

Furthermore, high slump concrete is used so that the complete volume of the hole is filled, including natural soil crevices and pockets and surface irregularities caused by the drilling operations. High slumps are also used where the concrete must displace drilling slurry.

Concrete slumps as high as 8 inches (203 mm) are sometimes needed where temporary casings are used to facilitate pile construction and are subsequently extracted as the concrete is placed.

Concrete is usually placed through a funnel hopper at the top of the element opening, but other approved methods, such as pumping or tremie, may also be used.

### TABLE 1808.8.1

#### MINIMUM SPECIFIED COMPRESSIVE STRENGTH f’c OF CONCRETE OR GROUT

<table>
<thead>
<tr>
<th>FOUNDATION ELEMENT OR CONDITION</th>
<th>SPECIFIED COMPRESSIVE STRENGTH, f’c</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Foundations for structures assigned to Seismic Design Category A, B or C</td>
<td>2,500 psi</td>
</tr>
<tr>
<td>2a. Foundations for Group R or U occupancies of light-frame construction, two stories or less in height, assigned to Seismic Design Category D, E or F</td>
<td>2,500 psi</td>
</tr>
<tr>
<td>2b. Foundations for other structures assigned to Seismic Design Category D, E or F</td>
<td>3,000 psi</td>
</tr>
<tr>
<td>3. Precast nonprestressed driven piles</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>4. Socketed drilled shafts</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>5. Micropiles</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>6. Precast prestressed driven piles</td>
<td>5,000 psi</td>
</tr>
</tbody>
</table>

For SI: 1 pound per square inch = 0.00689 MPa.
TABLE 1808.8.2. See below.

- This table lists the minimum concrete cover based on the type of foundation and the type of exposure.

1808.8.3 Placement of concrete. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-size foundation. Concrete shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water. Where depositing concrete from the top of a deep foundation element, the concrete shall be chuted directly into smooth-sided pipes or tubes or placed in a rapid and continuous operation through a funnel hopper centered at the top of the element.

- Placing concrete under water should be avoided wherever possible. The risk of segregation of the concrete mixture is much greater when depositing under water as opposed to air; however, when concrete must be placed under water, it should be done by any one of several accepted methods used for such construction, including tremie. Due care must be exercised in the concreting operations so as to avoid or minimize segregation of the mix and turbulence of the water.

   Generally, when concrete is to be placed under water, the mixture should be proportioned to provide a good plastic mix and high workability so that it will flow without segregation. The slump of the concrete should be 5 inches (127 mm) or greater. This desired consistency can be obtained by the use of rounded aggregates, a higher percentage of fines and entrained air. Cement content should be increased by 10 to 15 percent above the quantities required for similar mixtures placed in air to compensate for increases in water-cement ratios (see Chapter 19). In no case should the cement content be less than 600 pounds per cubic yard (355 kg/m³) of concrete.

1808.8.4 Protection of concrete. Concrete foundations shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

- Concrete for foundations should not be placed during rain, sleet or snow or in freezing weather unless adequate protection, as approved by the building official, is provided. Such protection, when required, is to be provided during the concreting operations and for a period of not less than five days thereafter. Rainwater or water from other sources must not be allowed to flow through freshly deposited concrete so as to increase the mixing water content or to damage the surface finish. For detailed information on materials, methods and procedures used in cold-weather operations, refer to ACI 306R, ACI 306.1 and Chapter 19.

1808.8.5 Forming of concrete. Concrete foundations are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require formwork. Where formwork is required, it shall be in accordance with Chapter 6 of ACI 318.

- Where earth cuts are used as the concrete form in foundation construction, the soil must have sufficient stiffness to maintain the desired shape and dimensions before and during concreting operations. In the event that the soil is deemed to be unstable for such purpose, the building official is to require that formwork be built in accordance with the provisions of ACI 318.

1808.8.6 Seismic requirements. See Section 1908 for additional requirements for foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.12.1 through

<table>
<thead>
<tr>
<th>FOUNDATION ELEMENT OR CONDITION</th>
<th>MINIMUM COVER</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Shallow foundations</td>
<td>In accordance with Section 7.7 of ACI 318</td>
</tr>
<tr>
<td>2. Precast non prestressed deep foundation elements</td>
<td></td>
</tr>
<tr>
<td>Exposed to seawater</td>
<td>3 inches</td>
</tr>
<tr>
<td>Not manufactured under plant conditions</td>
<td>2 inches</td>
</tr>
<tr>
<td>Manufactured under plant control conditions</td>
<td>In accordance with Section 7.7.3 of ACI 318</td>
</tr>
<tr>
<td>3. Precast prestressed deep foundation elements</td>
<td></td>
</tr>
<tr>
<td>Exposed to seawater</td>
<td>2.5 inches</td>
</tr>
<tr>
<td>Other</td>
<td>In accordance with Section 7.7.3 of ACI 318</td>
</tr>
<tr>
<td>4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing</td>
<td>2.5 inches</td>
</tr>
<tr>
<td>5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing</td>
<td>1 inch</td>
</tr>
<tr>
<td>6. Structural steel core within a steel pipe, tube or permanent casing</td>
<td>2 inches</td>
</tr>
<tr>
<td>7. Cast-in-place drilled shafts enclosed by a stable rock socket</td>
<td>1.5 inches</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm.
21.12.4. shall apply where not in conflict with the provisions of Sections 1808 through 1810.

Exceptions:

1. Detached one- and two-family dwellings of light-frame construction and two stories or less above grade plane are not required to comply with the provisions of ACI 318, Sections 21.12.1 through 21.12.4.

2. Section 21.12.4(a) of ACI 318 shall not apply.

This section first provides an important reference to Section 1908 for additional requirements concerning footings and foundations of buildings assigned to Seismic Design Category C, D, E or F. As specified in Section 1908.1.8, all footings and foundations in these structures require reinforcement except for footings supporting walls in detached one- and two-family dwellings three stories or less in height and constructed with stud-bearing walls. The concern is that plain concrete footings may be susceptible to damage, which can be reduced or avoided if reinforcement is provided. Foundation damage is obviously very expensive to repair and is better avoided.

The second paragraph of this section refers to provisions in ACI 318 concerning the design and construction of foundations of buildings assigned to high seismic design categories. Chapter 21 of ACI 318 is titled "Earthquake-resistant Structures," and Section 21.12 contains the requirements pertaining to foundations. Additionally, relatively small, detached dwellings classified as Seismic Design Category D or E are exempted from the referenced ACI 318 provisions. These structures would only need to comply with requirements for Seismic Design Category C.

1808.9 Vertical masonry foundation elements. Vertical masonry foundation elements that are not foundation piers as defined in Section 202 shall be designed as piers, walls or columns, as applicable, in accordance with TMS 402/ACI 530/ASCE 5.

This provision directs the code user to the definition of "Masonry foundation pier," which states that it has a height less than or equal to four times its thickness. Furthermore, the code user is referred to TMS 402/ACI 530/ASCE5 for the appropriate design requirements where the definition limits are exceeded. A vertical masonry element will be designed as a foundation pier, a pier, a wall or a column, depending on the dimensions.

SECTION 1809
SHALLOW FOUNDATIONS

1809.1 General. Shallow foundations shall be designed and constructed in accordance with Sections 1809.2 through 1809.13.

Sections 1809.2 through 1809.13 address requirements related to proper design and installation of shallow foundations.

1809.2 Supporting soils. Shallow foundations shall be built on undisturbed soil, compacted fill material or controlled low-strength material (CLSM). Compacted fill material shall be placed in accordance with Section 1804.5. CLSM shall be placed in accordance with Section 1804.6.

It is important that shallow foundations be built on undisturbed soil of known bearing value or properly compacted fill, with known bearing capacity. As an alternative to compacted fill, the code permits the use of CLSM (see commentary, Section 1804.6).

1809.3 Stepped footings. The top surface of footings shall be level. The bottom surface of footings shall be permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

The tops and bottoms of footings are required to be essentially level, with a slope of 1 unit vertical in 10 units horizontal (10-percent slope) permitted for the bottom of footings. Where the slope of the surface of the ground exceeds one unit vertical in 10 units horizontal (10-percent slope), footings are required to be stepped. Although not specifically mentioned, crack propagation at the joints should be considered when determining the overlapping and vertical dimensions of the steps.

1809.4 Depth and width of footings. The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the requirements of Section 1809.5 shall also be satisfied. The minimum width of footings shall be 12 inches (305 mm).

Footings are required to extend below the ground surface a minimum of 12 inches (305 mm). This is considered a minimum depth to protect the footing from movement of the soil caused by freezing and thawing in cold climates areas (see Section 1809.5 for general frost protection requirements).

1809.5 Frost protection. Except where otherwise protected from frost, foundations and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE 32; or
3. Erecting on solid rock.

Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. Assigned to Risk Category I, in accordance with Section 1604.5;
2. Area of 600 square feet (56 m²) or less for light-frame construction or 400 square feet (37 m²) or less for other than light-frame construction; and
3. Eave height of 10 feet (3048 mm) or less.
SOILS AND FOUNDATIONS

Shallow foundations shall not bear on frozen soil unless such frozen condition is of a permanent character.

- Shallow foundations must be placed on soil strata with adequate load-bearing capacity and at depths to which freezing cannot penetrate. In winter, frost action can raise the ground level (frost heave), whereas in springtime the same area will soften and settle back to its previous state. If foundations are built in soil strata that can freeze, then the heave or vertical movement of the ground, which is rarely uniform, can cause serious damage to buildings and other structures. Frost heave can become particularly aggravated in clay soils. Well-drained soils, such as sand and gravel, are not as susceptible to extensive movements.

Unless the exception applies, the foundation is to be protected from frost in accordance with this section. A common method of accomplishing this is by placing the footing bottom below the frost line. The “Frost line” is defined as the lowest level below the ground surface to which a temperature of 32°F (0°C) extends. The factors determining the depth of the frost line are air temperature and the length of time the temperature is below freezing [32°F (0°C)], as well as the ability of the soil to conduct heat and its level of thermal conductivity. Frost lines vary significantly throughout the country, ranging from 5 inches (127 mm) in the deep south to 100 inches (2540 mm) in the uppermost northern regions. The frost-free depth for shallow foundations is dependent on the frost line set for the particular locality of construction.

Another form of protection is the use of frost-protected shallow foundations (FPSF) in accordance with ASCE 32. This type of frost protection utilizes slab edge insulation to minimize heat loss at the slab edge. By retaining heat from the building in the ground, it has the effect of raising the frost line around the perimeter of the building.

Foundations are not to be placed on frozen soil because when the ground thaws, uneven settlement of the structure is apt to occur, thereby causing structural damage. This section does, however, permit footings to be constructed on permanently frozen soil. In permafrost areas, special precautions are necessary to prevent heat from the structure from thawing the soil beneath the foundation.

The exception to frost protection applies to low-risk structures, such as a detached garage. The permitted areas are intended to accommodate garage sizes that are commonly constructed. Note that the minimum footing depth of Section 1809.4 would apply where this exception is invoked.

1809.6 Location of footings. Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

- The bottoms of adjacent footings bearing on granular soil are to be located so that a line drawn between their closest edges would not be steeper than 30 degrees (0.52 rad) from the horizontal. The exceptions are where the soil surrounding the higher footing is laterally braced or retained as approved by the building official where engineering analysis shows that a greater slope can be tolerated. The purpose of this restriction is to provide conditions of safety against the possible influence of lateral and vertical soil pressures transmitted to the lower footing(s) by the loads of higher adjacent foundations (see Figure 1809.6).

```
For SI: 1 degree = 0.01745 rad.
```

Figure 1809.6
ISOLATED FOOTINGS
1809.7 Prescriptive footings for light-frame construction. Where a specific design is not provided, concrete or masonry-unit footings supporting walls of light-frame construction shall be permitted to be designed in accordance with Table 1809.7.

- This section allows prescriptive designs for concrete and masonry-footings supporting walls of light-frame construction when a specific design is not provided.

<table>
<thead>
<tr>
<th>NUMBER OF FLOORS SUPPORTED BY THE FOOTING</th>
<th>WIDTH OF FOOTING (inches)</th>
<th>THICKNESS OF FOOTING (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>8*</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

- a. Depth of footings shall be in accordance with Section 1809.4.
- b. The ground under the floor shall be permitted to be excavated to the elevation of the top of the footing.
- c. Interior stud-bearing walls shall be permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.
- d. See Section 1905 for additional requirements for concrete footings of structures assigned to Seismic Design Category C, D, E or F.
- e. For thickness of foundation walls, see Section 1807.1.6.
- f. Footings shall be permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.
- g. Plain concrete footings for Group R-3 occupancies shall be permitted to be 6 inches thick.

- Table 1809.7 provides prescriptive designs for footings supporting walls of light-frame construction when a specific design is not provided. The first column of the table is specifically intended to apply to the number of floors supported by the footing, not necessarily the number of stories of a building. This table provides for the minimum width and thickness of footings.

1809.8 Plain concrete footings. The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil or rock.

**Exception:** For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

- An isolated spread footing of square or rectangular shape is the most common type of shallow foundation used in building construction. Basically, the footing is a slab of concrete supporting a column or other concentrated load. The footing thickness or depth is generally a function of shear strength and is established by the more severe of two structural conditions. The thickness of a footing determined by one-way shear action may be compared to the shear in a beam where the vertical shear plane extends across the entire structural section. Thickness established by two-way shear action (punching shear) is based on the consideration that the column (or column pedestal) tends to punch through the footing and failure occurs as a fracture in the form of a truncated pyramid concentrated under the load (see Figure 1809.8). Foundation designs will normally result in concrete footing thicknesses that are greater than the minimum thicknesses prescribed in the code. The minimum thickness requirements for plain concrete footings are intended to provide adequate construction based on experience.

In compliance with the requirements of ACI 318, the edge thickness for plain concrete footings is not to be less than 8 inches (203 mm). In computing footing stresses (flexure, combined flexure, axial load and shear), however, the overall depth is to be 2 inches (51 mm) less than the actual thickness of the footing for all foundations on soil (see ACI 318 for design of plain concrete members) to allow for the irregularities of excavated earth surfaces and for possible contamination of the concrete by the soil in contact with the construction. The exception permits the edge thickness of plain concrete footings to be reduced to 6 inches (152 mm) for occupancies in Group R-3 of light-frame construction where the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall. For lightweight construction with the dimensional limitations given, shear stresses in the concrete will not usually govern the design thicknesses of such foundations. Plain concrete is not to be used for footings on piles.

1809.9 Masonry-unit footings. The design, materials and construction of masonry-unit footings shall comply with Sections 1809.9.1 and 1809.9.2, and the provisions of Chapter 21.

**Exception:** Where a specific design is not provided, masonry-unit footings supporting walls of light-frame
SOILS AND FOUNDATIONS

Construction shall be permitted to be designed in accordance with Table 1809.7.

 Sections 1809.9.1 and 1809.9.2 provide general requirements for the construction of solid masonry footings. This type of foundation was widely used until the middle of the last century when it was replaced by steel or wood grillage and eventually by more economical plain and reinforced concrete foundations. At that time, masonry foundations were built of either stone cut to specific sizes or of rubble stone of random sizes bonded together with mortar. Masonry footings under columns were usually constructed as high piers in the shape of truncated pyramids. While stone footings may still exist in very old structures, they are extinct in modern construction. Today, although scarcely used, masonry footings may be constructed of hard-burned brick in cement mortar to support the walls of lightweight construction, such as for low residential buildings. Brick footings are usually set on a full bed of mortar spread upon the earth or a grout sill. Stepped footing courses are recommended to be built with the brick units laid on edge, which are capable of resisting a greater transverse stress than flat courses.

 The exception applies to footings supporting walls of light-frame construction when a specific footing design is not provided. The purpose of Table 1809.7 is to specify footing sizes and depths that can be used to safely support walls of light-frame construction. The table is based on anticipated loads on foundations due to wall, floor and roof systems.

1809.9.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.8. Type M is a high-strength mortar suitable for general use and, in particular, where maximum masonry compressive strength is required. It is also suitable for unreinforced masonry below grade and in construction that is in contact with earth. Type S is a general-use mortar where high lateral strength of masonry is required. It is specifically recommended for use in reinforced masonry.

 Projections of footings beyond a wall, pier or column base are not to exceed dimensions by more than one-half the depth of the foundation. For example, a footing supporting a wall and consisting of three courses of brick with a depth of about 8 inches (203 mm) is not to project more than 4 inches (102 mm) (1/4 by 8 inches) beyond the base of the wall. This is to keep the shearing stresses in the footing within a safe limitation.

 Masonry footings are not to be less than 8 inches (203 mm) wider than the thickness of the supported wall so as to provide adequate distribution of the wall load to the load-bearing soil.

1809.9.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be 1 1/2 inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

 Foundation walls that entail stepping back successive courses of brick (called racking) are not to have horizontal offsets exceeding 1 1/2 inches (38 mm) from the face of the course below. If the steps are made in double course increments, then the offsets must not exceed 3 inches (76 mm).

 Sometimes, wide footings are required so as to exceed the safe load-bearing capacity of the supporting soil. Since footing projections beyond a wall are limited in dimension (see Section 1809.9.1), the wall must inevitably be stepped out to provide the proper base on the footing (see Figure 1809.9.2).

![Figure 1809.9.2 BRICK FOOTING OFFSET](image)

For SI: 1 inch = 25.4 mm.
1809.10 Pier and curtain wall foundations. Except in Seismic Design Categories D, E and F, pier and curtain wall foundations shall be permitted to be used to support light-frame construction not more than two stories above grade plane, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.

2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 37/8 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).

3. Piers shall be constructed in accordance with Chapter 21 and the following:

   3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.

   3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

   Exception: Unfilled hollow piers shall be permitted where the unsupported height of the pier is not more than four times its least dimension.

   3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.

4. The maximum height of a 4-inch (102 mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.

5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305 mm) for hollow masonry.

Pier and curtain wall foundation systems are now and have been a very popular method of construction for many years, particularly throughout the eastern United States. The origin of the design is seen in many precolonial wood-frame buildings. Pier and curtain wall foundations are only permitted to support structures of light-frame construction (wood or light-gage steel framing members) not more than two stories in height and assigned to Seismic Design Category A, B or C. Seismic detailing requirements for higher seismic design categories have not yet been developed. The term "pier" used in this application means "pilaster" rather than a type of foundation. The term "curtain wall" refers to minimum 4-inch-thick (102 mm) masonry load-bearing walls. The provisions apply to simple wood-frame buildings where the combined loads are minimal.

This type of foundation system is also addressed in Section R404.1.5.3 of the International Residential Code® (IRC®), which includes an accompanying figure illustrating the code requirements. This IRC figure is reproduced here as Figure 1809.10.

1809.11 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

Steel grillage footings were extensively used during the latter part of the last century, but the development and use of reinforced concrete foundations have made this type of construction all but obsolete. They are, however, still used for underpinning purposes. There are many steel grillage footings in existence in old buildings.

A typical grillage footing consists of two or more tiers of steel beams (usually I-sections) with each tier placed at right angles to the one below it. The beams in each tier are usually held together by a system of bolts and pipe spacers. The beams should be clean and unpainted, and the whole system completely filled and encased in concrete with at least 6 inches (152 mm) of cover on the bottom and 4 inches (102 mm) at all other points. In lieu of concrete (other than the encasement), the spaces between the steel beams may be filled with cement grout.

1809.12 Timber footings. Timber footings shall be permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level, or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported upon treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AF&PA NDS.

The use of timber footings is allowed only in Type V construction, or as otherwise approved by the building official. Such footings are commonly built as grillages of heavy timbers, using large sections, such as railroad ties.

If the footings are to be constructed at depths above the water table, they are to be given a preservative treatment by pressure processes to protect the materials from decay, fungi and harmful insects. Pressure treatment is to be in accordance with the requirements of the U1 standard of the American Wood Protection Association (AWPA). AWPA U1, Specification A, Use Category 4B, refers to sawn wood products that are used in contact with the ground. Except for use on cut surfaces, brush or spray applications of preservatives are not acceptable methods of treatment.

Preservative treatment by the pressure process within the limitations specified in AWPA U1 should not significantly affect the strength of the wood. Part of the process, however, involves the conditioning of...
timbers before treatment. Timbers conditioned by steaming or boiling under vacuum can suffer significant reductions in strength. This condition is recognized in the American Forest and Paper Association National Design Specification® (AF&PA NDS) by the use of the untreated factor $C_u$.

The design values for treated round timber piles given in this specification are adjusted to compensate for strength reductions because of conditioning prior to treatment. It is recommended that the values given for the several species of wood be used in the design of treated timber footings. Design values given in other tables contained in the AF&PA referenced standard are for general structural purposes using untreated lumber. While the section of this specification for pressure-preservative treatment stipulates that the design values apply to wood products that are pressure impregnated by an approved process, it is not apparent that reductions have been made to allow for possible strength loss in the wood because of the treatment process.

Untreated timber may be used when the footings are completely embedded in soil below the water level. Experience has shown that timber foundations permanently confined in water will stay sound and durable indefinitely. Wood submerged in fresh water cannot decay because the necessary air is excluded. It is not uncommon, however, to have changing ground-water levels because of changes in adjacent drainage systems and a variety of other subterranean

---

**Figure 1809.10**

FOUNTION WALL CLAY MASONRY CURTAIN WALL WITH CONCRETE MASONRY PIERS

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
conditions. Precautions should be taken so that untreated timber footings are placed at depths sufficiently below the groundwater level such that small drops in the water level will not expose the footings to air. Otherwise, decay will set in, causing settlements and possible structural failures.

The code also prescribes that compressive stresses perpendicular to grain in untreated timber footings supported upon piles are not to exceed 70 percent of the allowable stresses specified for the species and grade of timber given in the AF&PA referenced standard.

1809.13 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F, individual spread footings founded on soil defined in Section 1613.3.2 as Site Class E or F shall be interconnected by ties. Unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger footing design gravity load times the seismic coefficient, $S_{D}$ divided by 10 and 25 percent of the smaller footing design gravity load.

- This section requires that spread footings on soft soil profiles be interconnected by ties when they support structures assigned to Seismic Design Categories D and higher. The purpose of this section is to preclude excessive movement of one column or wall with respect to another. One of the prerequisites of adequate structural performance during an earthquake is that the foundation of the structure acts as a unit. This is typically accomplished by tying together the individual footings with ties capable of carrying, in tension or compression, the smaller of 10 percent of the larger footing gravity load multiplied by $S_{D}$ or 25 percent of the smaller footing design gravity load. $S_{D}$ is the design spectral response acceleration at short periods, as determined in Section 1613.3.4. This tie can be provided by concrete floor slabs or tie beams. The differential movement of the foundation should not exceed that included in the design of the seismic-force-resisting system.

SECTION 1810
DEEP FOUNDATIONS

1810.1 General. Deep foundations shall be analyzed, designed, detailed and installed in accordance with Sections 1810.1 through 1810.4.

- This section sets forth the general rules for analyzing, designing, detailing and installing deep foundations.

1810.1.1 Geotechnical investigation. Deep foundations shall be designed and installed on the basis of a geotechnical investigation as set forth in Section 1803.

- A foundations investigation is mandatory when deep foundations are used. Such investigations are needed to define as accurately as possible the subsurface conditions of soil and rock materials, establish the soil and rock profiles across the construction site and locate the groundwater table. Sometimes it may also be necessary to determine specific soil properties, such as shear strength, relative density, compressibility and other such technical data required for analyzing subsurface conditions. Foundation investigations may also be used to render such valuable data as information on existing construction at the site or on neighboring properties (including boring and test records), the type and condition of the existing structures, their ages, the type of foundations used and performance over the years. Knowledge of existing deleterious substances in the soils that could affect the durability (as well as the performance) of foundation elements, data on geologic conditions at the site as well as a history of seismic activity are also important.

1810.1.2 Use of existing deep foundation elements. Deep foundation elements left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the elements are sound and meet the requirements of this code. Such elements shall be load tested or redriven to verify their capacities. The design load applied to such elements shall be the lowest allowable load as determined by tests or redriving data.

- After the demolition of an existing building, any deep foundation elements remaining in place cannot be reused to support a new structure unless sufficient and reliable information is provided to the building official showing that the new loads to be imposed will be adequately supported by the existing deep foundation elements. This requirement is necessary because of the lack of adequate soil data and technical information on the material used and the unavailability of pile-driving records made during the construction of the existing deep foundation. The current condition of the deep foundation element is not known, since it may have deteriorated over time, possibly reducing its load capacity. Deep foundation capacities may be determined by load tests, or the elements may be retracted and redriven to verify their load capacities.

1810.1.3 Deep foundation elements classified as columns. Deep foundation elements standing unbraced in air, water or fluid soils shall be classified as columns and designed as such in accordance with the provisions of this code from their top down to the point where adequate lateral support is provided in accordance with Section 1810.2.1.

Exception: Where the unsupported height to least horizontal dimension of a cast-in-place deep foundation element does not exceed three, it shall be permitted to design and construct such an element as a pedestal in accordance with ACI 318.

- The section addresses the condition where deep foundation elements are not laterally supported by soil and, therefore, must be designed as columns. This design condition applies from the top of the ele-
SOILS AND FOUNDATIONS

tment to a point at which the soil can be assumed to provide lateral support as described in Section 1810.2.1. The exception addresses concrete foundation elements with a height no greater than three times the least horizontal dimension, recognizing that these elements can be designed as pedestals under ACI 318.

1810.1.4 Special types of deep foundations. The use of types of deep foundation elements not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such elements. The allowable stresses for materials shall not in any case exceed the limitations specified herein.

Deep foundations are generally identified according to the materials used (concrete, steel or wood) or the methods of construction or installation. While the most commonly used types of deep foundations are addressed in this section, there are many variations of deep foundation types used in construction, including some special or proprietary types that are beyond the scope of the code.

However, while it is not the intent to preclude the use of such special or proprietary types of deep foundations, it is necessary to substantiate their structural performance by submitting test data, calculations, information on structural properties and load capacity and installation procedures to the building official for approval.

1810.2 Analysis. The analysis of deep foundations for design shall be in accordance with Sections 1810.2.1 through 1810.2.5.

This section provides requirements applicable to the analysis of deep foundations, such as lateral support, stability, settlement, lateral loads and group effects.

1810.2.1 Lateral support. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to prevent buckling of deep foundation elements and to permit the design of the elements in accordance with accepted engineering practice and the applicable provisions of this code.

Where deep foundation elements stand unbraced in air, water or fluid soils, it shall be permitted to consider them laterally supported at a point 5 feet (1524 mm) into stiff soil or 10 feet (3048 mm) into soft soil unless otherwise approved by the building official on the basis of a geotechnical investigation by a registered design professional.

The primary concern that is addressed in this section is the consideration of slenderness effects in the design of a compression element. Experience with deep foundation performance under loaded conditions has shown that elements embedded in earth, including even relatively soft and compressible clays, exhibit lateral restraint that is sufficient to prevent buckling.

Deep foundation elements that are driven into fluid soils, such as saturated silts, as well as portions of deep foundation elements that project above the supporting soil are not laterally supported and, therefore, may be susceptible to buckling. Under such conditions, these elements must be designed as columns in accordance with the applicable provisions of the code.

This section allows the assumption of lateral support at a prescribed embedment without requiring confirmation by a soils investigation. The embedment required is the distance into either stiff soil or soft soil (not necessarily the distance below the ground surface). Note that, while the terms "stiff soil" and "soft soil" are not defined, they are generally consistent with the terms used in Section 1613.3.2 for Site Classes D and E. This provision essentially provides a starting point for analyzing the fixity of the deep foundation element.

1810.2.2 Stability. Deep foundation elements shall be braced to provide lateral stability in all directions. Three or more elements connected by a rigid cap shall be considered braced, provided that the elements are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-element group in a rigid cap shall be considered to be braced along the axis connecting the two elements. Methods used to brace deep foundation elements shall be subject to the approval of the building official.

Deep foundation elements supporting walls shall be placed alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the foundation elements are adequately braced to provide for lateral stability.

Exceptions:

1. Isolated cast-in-place deep foundation elements without lateral bracing shall be permitted where the least horizontal dimension is no less than 2 feet (610 mm), adequate lateral support in accordance with Section 1810.2.1 is provided for the entire height and the height does not exceed 12 times the least horizontal dimension.

2. A single row of deep foundation elements without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in building height, provided the centers of the elements are located within the width of the supported wall.

A group of deep foundation elements designed to support a common load and, as may be required, to resist horizontal forces, must be braced or rigidly tied together to act as a single structural unit that will provide lateral stability in all directions. Deep foundation elements that are connected by a rigid, reinforced concrete pile cap are deemed to be braced construction that serves the intent of this provision.

Three or more deep foundation elements are generally used to support a building column load or other isolated concentrated load. In a three-pile group, lateral stability is provided by requiring that the ele-
ments are located such that they will not be less than 60 degrees (1.0 rad) apart as measured from the centroid of the group in a radial direction. For stability of deep foundation elements supporting a wall structure, the elements are braced by a continuous rigid footing and are alternately staggered in two lines that are at least 1 foot (305 mm) apart and symmetrically located on each side of the center of gravity of the wall. Other approved deep foundation arrangements may be used to support walls, provided that the elements are adequately braced and the lateral stability of the foundation construction is ensured.

Exception 1 addresses the use of isolated elements without lateral bracing. To qualify elements must have widths (or diameters) of at least 2 feet (610 mm) and be proportioned so that the height (length) of the element is no more than 12 times the least lateral dimension. This is an empirical requirement that is intended to offset the concerns that typically require consideration in more slender elements.

For one- and two-family dwellings as well as lightweight construction that does not exceed two stories above grade plane or 35 feet (10 668 mm) in height, Exception 2 permits a single row of elements, rather than the minimum 1 foot (305 mm) offset, provided the deep foundation elements are located within the width of the supported wall. In this case, stability of the group is theoretically afforded only in the direction of the line of the deep foundation elements. However, in this limited group of relatively lighter buildings, the cross walls, floor slabs and other structural components are assumed to provide some degree of lateral stability to the deep foundation system.

**1810.2.3 Settlement.** The settlement of a single deep foundation element or group thereof shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity.

- A settlement analysis is performed to design a deep foundation system that will maintain the stability and structural integrity of the supported building or structure. Foundation systems that suffer serious settlements, particularly differential settlements, can cause structural damage to the supported structure as well as to the foundation itself.

  The settlement analysis of an individual element is a complex procedure. In comparison, the analysis of a group of elements is even more complex because of the overlapping soil stresses caused by closely spaced piles. Analytical procedures vary with the type of elements and especially with the type of soil.

  Settlements are of two basic types: immediate settlements are those that occur as soon as the load is applied and usually take place within a period of less than seven days; consolidation settlements are time dependent and take place over a long period of time. All cohesionless soils, such as granular materials consisting of sand, gravel or a mixture of both, which have a large coefficient of permeability (rapid draining properties), undergo immediate settlements. All fine-grained, saturated, cohesive soils, such as clays, undergo time-dependent consolidation settlements. Settlement analysis would generally include cases involving piers and end-bearing piles on rock or hard soils as well as friction-type piles in both granular and cohesive soils. Load tests are often performed to provide data that will aid the settlement analysis.

  The code places no specific limitation on the amount of settlement; instead, it states that the effects of any settlement, particularly differential settlements, should not be harmful to the structure. Thus, the tolerable settlement is largely dependent on the type of structure being considered. Guidance on tolerable settlements can be found in engineering textbooks, such as *Soil Mechanics in Engineering Practice* (Terzaghi and Peck), *Foundation Analysis and Design* (Bowles) and *Foundation Engineering* (Hanson, Peck and Thornburn). As a rule, these references indicate that a total settlement of 1 inch (25 mm) is acceptable for the majority of structures, while other structures can tolerate even greater settlements without distress. On the other hand, a more restrictive settlement criterion can be necessary based on the needs of a specific structure. Current engineering practice is often based on an allowable total settlement of 1 inch (25 mm) with the objective of controlling the differential settlements to 0.06 inch (19 mm) or less. The effects of differential settlements need to be considered in the structural design as a self-straining force, \( T \), when designing for the load combinations of Section 1605.

**1810.2.4 Lateral loads.** The moments, shears and lateral deflections used for design of deep foundation elements shall be established considering the nonlinear interaction of the shaft and soil, as determined by a registered design professional. Where the ratio of the depth of embedment of the element to its least horizontal dimension is less than or equal to six, it shall be permitted to assume the element is rigid.

- This section addresses miscellaneous issues unique to the seismic design of deep foundations. If the length is less than or equal to six times the least horizontal dimension of the element, it can be assumed to be rigid. Then moments, shears and lateral deflections can be calculated accordingly. Where the length exceeds six times the least horizontal dimension, the nonlinear interaction of the element and soil effects are to be included in the analysis. The effect of abrupt changes in soil deposits, such as changes from soft to firm or loose to dense soils, should be included in the analysis.

**1810.2.4.1 Seismic Design Categories D through F.** For structures assigned to *Seismic Design Category* D, E or F, deep foundation elements on *Site Class* E or F sites, as determined in Section 1613.3.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-foundation-
structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure.

**Exception:** Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section 1810.3.8.3.3.

2. Cast-in-place deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the element and detailed in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318 as required by Section 1810.3.9.4.2.2.

The first paragraph of this section requires special consideration of flexural loads on deep foundation elements due to earthquake motions. The following discussion taken from the NEHRP Provisions commentary, Section 7.5.4, provides justification for these requirements:

Special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to element design where portions of the deep foundation elements are supported in soils, such as loose granular materials or soft soils that are susceptible to large deformations or strength degradation. Severe pile bending problems can result from various combinations of soil conditions during strong ground shaking. For example:

- Soil settlement at the pile-cap interface either from consolidation of soft soil prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.

- Large deformations, a reduction in strength or both, resulting from liquefaction of loose, granular materials, can cause bending or conditions of free-standing columns.

- Large deformations in soft soils can cause varying degrees of element bending. The degree of bending will depend upon thickness and strength of the soft soil layer(s) and the properties of the soft/stiff soil interface(s).

The designer needs to consider the variation in soil conditions and driven element lengths in providing for pile ductility at potential high curvature interfaces. Interaction between the geotechnical and structural engineers is essential.

It is prudent to design deep foundation elements to remain functional during and following earthquakes in view of the fact that it is difficult to repair foundation damage. The desired foundation performance can be accomplished by proper selection and detailing of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves.

**1810.2.5 Group effects.** The analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. The analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element.

- This section also prescribes conditions under which group effects on the nominal pile strength, lateral as well as vertical, need to be considered in the analysis.

**1810.3 Design and detailing.** Deep foundations shall be designed and detailed in accordance with Sections 1810.3.1 through 1810.3.12.

- This section provides requirements and guidance for the design and detailing of deep foundation elements. This includes material-specific criteria, allowable loads, dimensions, splices, pile caps, grade beams and seismic ties.

**1810.3.1 Design conditions.** Design of deep foundations shall include the design conditions specified in Sections 1810.3.1.1 through 1810.3.1.6, as applicable.

- This section covers design methods for concrete elements, guidance for composite elements, effects of mislocation, driven piles, helical piles and casings.

**1810.3.1.1 Design methods for concrete elements.** Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 or the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of concrete deep foundation elements shall use the load combinations of Section 1605.2 and approved strength design methods.

- This provision states what is typical in current practice for foundation design. For decades structural concrete design has primarily been based on the strength design method. However, there is also a long tradition of using simple allowable stress design approaches for the proportioning of deep foundation elements (for both soil-foundation behavior and structural design). This section recognizes allowable stress design for elements that are concentrically loaded and laterally supported.

**1810.3.1.2 Composite elements.** Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section of the composite assembly shall satisfy the applicable requirements of this code, and the maximum allowable load in each section shall be limited by the structural capacity of that section.

- Composite elements are made of two or more sections of different materials or of different types that
are spliced together to form a single deep foundation element. These elements are typically used when significant lengths are needed. Due to economic considerations and problems with splicing, composite elements are now used less frequently. An illustration of typical composite elements is shown in Figure 1810.3.1.2.

This section gives the general requirement that each section of a composite element must comply with the applicable provisions for that element type as well as the material comprising that section. Furthermore, the element's overall load capacity must be based on the most restrictive permitted value for all of the sections.

![TYPICAL COMBINATION

Figure 1810.3.1.2 COMPOSITE ELEMENTS

1810.3.1.3 Mislocation. The foundation or superstructure shall be designed to resist the effects of the mislocation of any deep foundation element by no less than 3 inches (76 mm). To resist the effects of mislocation, compressive overload of deep foundation elements to 110 percent of the allowable design load shall be permitted.

Because of subsurface obstructions or other reasons, it is sometimes necessary to offset deep foundation elements a small distance from their intended locations or they may be driven out of position. In such cases, the load distribution in a group of elements may be changed from the design requirements and cause some of the elements to be overloaded. This section requires that the maximum compressive load on any deep foundation element caused by mislocation should not exceed 110 percent of the allowable design load. Elements exceeding this limitation must be extracted and installed in the proper location or other approved remedies must be applied, such as installing additional elements to balance the group.

1810.3.1.4 Driven piles. Driven piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

- Precast elements must be properly designed to resist the stresses induced both by handling and driving operations, and later imposed by service loads. Care must be given during handling and installation to minimize or avoid possible damage to these elements, such as cracking, crushing or spalling.

1810.3.1.5 Helical piles. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads.

- See the definition of "Helical pile" in Section 1802.1. This section gives only general guidance to design in accordance with accepted engineering practice. Helical pile systems having ICC Evaluation Service reports can be viewed at www.icc-es.org (also see ICC Evaluation Service Acceptance Criteria 358).

1810.3.1.6 Casings. Temporary and permanent casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Where a permanent casing is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1810.3.2.5. Horizontal joints in the casing shall be spliced in accordance with Section 1810.3.6.

- Because a casing is exposed to earth and water pressures, this section requires that casings are of adequate strength to resist serious damage or collapse and to maintain sufficient water tightness so as to prevent any foreign materials from entering during concrete placement (see Figure 1810.3.1.6 for an illustration of steel casings). Steel pipe casings driven with a mandrel can be torn or otherwise damaged because of underground obstructions, such as rock crevices. If this happens, the water tightness and structural integrity of the deep foundation element may be affected. Sometimes the buildup of ground pressures during driving may cause pile casings to squeeze or even collapse after the mandrel has been withdrawn.

1810.3.2 Materials. The materials used in deep foundation elements shall satisfy the requirements of Sections 1810.3.2.1 through 1810.3.2.8, as applicable.

- This section specifies minimum requirements for concrete, prestressing steel, structural steel timber, etc., for use in deep foundations.
1810.3.2.1 Concrete. Where concrete is cast in a steel pipe or where an enlarged base is formed by compacting concrete, the maximum size for coarse aggregate shall be \( \frac{3}{4} \) inch (19.1 mm). Concrete to be compacted shall have a zero slump.

This section limits the aggregate size in cased elements and enlarged base elements (see Section 1810.4.7). Concrete materials generally are to comply with the applicable requirements of Chapter 19. Coarse aggregate materials used in the concrete mix are not to exceed \( \frac{3}{4} \) inch (19.1 mm) in size. Concrete that will be compacted must have a zero slump in order to provide a stiff mix capable of being compacted by a heavy drop weight.

1810.3.2.1.1 Seismic hooks. For structures assigned to Seismic Design Category C, D, E or F, the ends of hoops, spirals and ties used in concrete deep foundation elements shall be terminated with seismic hooks, as defined in ACI 318, and shall be turned into the confined concrete core.

In structures that have moderate or high seismic risk, transverse reinforcement confines the concrete core of deep foundation elements and provides lateral supports for longitudinal bars. The requirement to terminate hoops, spirals and ties with seismic hooks parallels requirements in Chapter 21 of ACI 318.

1810.3.2.1.2 ACI 318 Equation (10-5). Where this chapter requires detailing of concrete deep foundation elements in accordance with Section 21.6.4.4 of ACI 318, compliance with Equation (10-5) of ACI 318 shall not be required.

In deep foundation elements the axial compression is limited by the capacity of the soil-foundation interface. This is also reflected by the allowable stresses for these elements which are limited to a smaller percentage of the concrete compressive strength. The purpose of ACI 318 Equation (10-5) is to provide significant residual compressive strength for concentrically loaded spiral columns subjected to very large axial compression. The amount of spiral reinforcement required by Equation (10-5) is intended to provide additional load-carrying strength for concentrically loaded columns equal to or slightly greater than the strength lost when the shell spalls off. The concrete cover required for uncased deep foundation elements is much greater than that for columns. As a result, application of Equation (10-5) to deep foundation elements results in amounts of transverse reinforcement that are unwarranted and often unplaceable. The primary concern in providing transverse confinement reinforcement for deep foundations is flexural ductility. Because proper design for deep foundation elements differs from that for columns in several respects, this provision relaxes an overly restrictive code requirement.

1810.3.2.2 Prestressing steel. Prestressing steel shall conform to ASTM A 416.

Rods, strands or wires conforming to the requirements of ASTM A 416 are used as prestressing steel in the manufacture of precast, prestressed concrete elements of both the pretensioned and the post-tensioned type. The most commonly used prestressing steel is the seven wire, uncoated, stress-relieved strand.

1810.3.2.3 Structural steel. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.

The materials used in the manufacture of steel piles must comply with requirements of one of the listed ASTM standards. While H-piles are ordinarily made with steel materials conforming to ASTM A 36 or ASTM A 572 require-
ments, the employment of other standards is not excluded, provided it can be shown that such steels meet the applicable chemical and mechanical properties of one of the listed specifications to establish the suitability of the material for pile use.

Welded or seamless pipe piles must conform to ASTM A 252 requirements.

1810.3.2.4 Timber. Timber deep foundation elements shall be designed as piles or poles in accordance with AF&PA NDS. Round timber elements shall conform to ASTM D 25. Sawn timber elements shall conform to DOC PS-20.

- Round timber elements are best suited as friction-type piles. They are not recommended to be driven through dense gravel, boulders or till, or for end-bearing piles on rock. While timber elements do not have load capacities of structural steel or concrete elements, they are probably the most commonly used type of deep foundation element throughout the United States, mainly because of their availability, ease of handling and, sometimes, for economic reasons.

Timber elements are shaped from tree trunks and their lengths are dependent on the heights that the various species of trees used for making piles will grow. Timber piles are made as tapered sections because of the natural taper of tree trunks.

Round timber piles are usually made from Southern pine in lengths up to about 80 feet (24 384 mm) and from Pacific Coast Douglas fir in lengths up to about 125 feet (38 100 mm). Other species commonly used for piles are red oak and red pine. Timber piles that are 40 feet (12 192 mm) to 60 feet (18 288 mm) in length are common, but longer lengths cannot be obtained economically in all areas of the country.

Untreated timber piles that are embedded below the permanent ground-water level (fresh water only) may last indefinitely. Under conditions where piles are required to extend above ground-water level but still remain totally embedded in the ground, the untreated wood material may be subject to decay. In cases where untreated timber piles extend above the ground surface into the air, they are exposed to decay and insect attack. The durability of timber piles is best served by applying treatment with an approved preservative.

This section requires timber deep foundation elements to be designed in accordance with the AF&PA NDS. Section 6 of AF&PA NDS provides the appropriate design values, different species of timber piles and the applicable adjustment factors. While timber piles have to be designed in accordance with the AF&PA NDS, they are usually only considered for design loads of 10 (89 kN) to 50 tons (445 kN).

One of the significant problems associated with timber pile installations is the possibility of damage caused by overdriving. Overdriving of wood piles may cause failure by bending, browning or brooming at the butt end or splitting or breaking along the pile section (see Figure 1810.4.1.5). Another problem is the difficulties encountered when splicing timber piles to achieve greater lengths.

The ASTM D 25 standard referenced in this section covers the physical characteristics of treated and untreated round timber piles. Essentially, the standard divides timber piles into two classifications: friction and end-bearing piles. The dimensional requirements for each classification are tabulated, giving the minimum circumference requirements for pile heads and the corresponding smaller tip circumference based on pile taper and lengths between 20 (6096 mm) and 120 feet (36 576 mm), measured in increments of 10 feet (3048 mm). The ASTM D 25 standard also includes requirements for the quality of wood, tolerances on pile straightness, twist of grain, knots, holes, scars, wood checks, shakes, splits and other necessary information. However, the requirements stated in the standard do not relate to the several species of wood used for making timber piles, nor to their relative strengths. A timber pile with typical dimensions from ASTM D 25 is shown in Figure 1810.3.2.4.

1810.3.2.4.1 Preservative treatment. Timber deep foundation elements used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber elements will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention...
shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber elements and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber elements. Preservative-treated timber elements shall be subject to a quality control program administered by an approved agency. Element cutoffs shall be treated in accordance with AWPA M4.

- For a general discussion on the need for treating wood with preservatives, see the commentary to Sections 1809.12,1810.3.2.4 and 1810.3.2.5. This section of the code specifically requires that the preservative treatment of round timber piles and sawn timber piles by pressure processes conforms to the applicable requirements of AWPA U1. Round timber pile requirements are covered in Commodity Specification E. Use Category 4C refers to conditions of very severe ground contact. Sawn timber piles are covered under Commodity Specification A and Use Category 4B, which refer to ground contact in severe environments. Note that round timber piles conform to ASTM D 25, while sawn timber piles comply with the grading rules of the species for strength.

- This section also requires that, for timber piles subjected to saltwater exposure, the treatment of piles with water-borne preservatives and creosote complies with quality control procedures administered by an approved agency. Guides used in establishing these procedures are those contained in the American Wood Preservers’ Bureau (AWPB) publications such as:

  - MP1, Quality Control and Inspection Procedures for Dual Treatment of Marine Piling Pressure Treated with Waterborne Preservatives and Creosote for Use in Marine Waters;
  - MP2, Quality Control and Inspection Procedures for Marine Piling Pressure Treated with Creosote for Use in Marine Waters; or
  - MP4, Quality Control and Inspection Procedures for Marine Piling Pressure Treated with Waterborne Preservatives for Use in Marine Waters.

- The field treatment of cuts and injuries to timber piles, including the treatment at pile cutoffs, with preservatives made for applications of creosote and creosote mixtures or with water-borne preservatives is covered by AWPA M4.

1810.3.2.5 Protection of materials. Where boring records or site conditions indicate possible deleterious action on the materials used in deep foundation elements because of soil constituents, changing water levels or other factors, the elements shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the elements so as not to be rendered ineffective by installation. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

- Deep foundation elements are often exposed to the deteriorating effects of biological, chemical and physical actions caused by a hostile underground environment that may exist at the time of their installation or that may later develop at the site. Under such conditions, deep foundation element materials must be properly protected to ensure their expected durability.

- The problems associated with element durability relate directly to the type of pile materials used. For example, concrete elements that are entirely embedded in undisturbed soil are deemed to be permanent installations. The level of the ground-water table is generally not a factor affecting the durability of concrete elements. Ground water that contains deleterious substances and readily flows through disturbed or granular soils, such as sand and gravel (regardless of the level of the water table), can have a deteriorating effect on concrete piles. Concrete elements embedded in fine-grained, impervious soils, such as clay, generally are not adversely affected by ground water containing harmful substances. Concrete can also be affected by exposure to soils having a high sulfate content, unless Type II or V Portland cement is used in making the concrete mixture.

- Concrete piles installed in saltwater, such as for buildings or other structures in waterfront construction, are subject to chemical action from polluted waters, frost action on porous concrete, spalling and rusting of steel reinforcement. Spalling action may become particularly critical under tidal conditions where alternate wetting and drying of the concrete occurs in conjunction with cycles of freezing and thawing. Generally, concrete can be protected from damage by such adverse conditions with the use of special cements, dense concrete mixtures rich in cement content, adequate concrete cover over the reinforcement, air entrainment, suitable concrete admixtures or special surface coatings.

- Elements made of steel materials and embedded entirely in undisturbed soil (regardless of its type) are not significantly affected by corrosion due to oxidation, mainly because undisturbed soil is so deficient in oxygen that progressive corrosion is repressed. However, steel may be subject to serious corrosion and structural deterioration where ground water contains deleterious substances from coal piles, alkali soils, active cinder fills, chemical waste from manufacturing operations, etc. Under such conditions, steel piles may be protected by encasement in concrete or by applying protective coatings, such as coal-tar or other suitable materials. Steel piles installed in saltwater or exposed to a saltwater environment can corrode severely and should be protected by encasement in concrete or by the application of approved coatings. Elements that extend above ground level and are exposed to air should be painted in the same way as any type of structural steel construction to prevent rusting. Corrosion of load-bearing steel can also occur because of electrolytic action, and in such cases, cathodic protection may be required.

- Timber piles totally embedded in the earth below the low point of ground-water level or entirely sub-

18-52 2012 INTERNATIONAL BUILDING CODE® COMMENTARY
merged in fresh water will last indefinitely without preservative treatment. However, timber piles that extend above the ground-water level or are exposed to air or saltwater are subject to decay as well as attacks by insects and marine borers. The piles may also be subjected to damage by the percolation of ground water heavily charged with alkali and acids. Under such conditions, timber piles must be pressure treated with preservatives in accordance with AWPA U1 listed in Chapter 35.

Generally, at any site where piles are to be installed and where the soil is suspect or there is sufficient evidence of an adverse underground environment, a soils investigation should be conducted to determine the need and method to protect the piles against possible deterioration.

1810.3.2.6 Allowable stresses. The allowable stresses for materials used in deep foundation elements shall not exceed those specified in Table 1810.3.2.6.

- This section refers the code user to the table of allowable stresses in order to identify the correct values that apply to various types of deep foundations. Note that Section 1810.1.4 allows “special types of piles” using the allowable stresses for materials that are specified herein.

TABLE 1810.3.2.6. See below.

- This table provides a complete list of the relevant allowable stresses for deep foundation element materials including concrete, reinforcing steel and structural steel.

1810.3.2.7 Increased allowable compressive stress for cased cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:

1. The design shall not use the casing to resist any portion of the axial load imposed.
2. The casing shall have a sealed tip and be mandrel driven.
3. The thickness of the casing shall not be less than manufacturer’s standard gage No.14 (0.068 inch) (1.75 mm).
4. The casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.
5. The ratio of steel yield strength ($f_y$) to specified compressive strength ($f'_c$) shall not be less than six.
6. The nominal diameter of the element shall not be greater than 16 inches (406 mm).

- For cased cast-in-place concrete elements formed by driving permanent steel casings, the allowable design compressive stress in Table 1810.3.2.6 is generally not to exceed $0.33 f'_c$. When the permanent casing complies with the requirements of this section, the allowable concrete compressive stress may be increased to $0.40 f'_c$. The basis for this increase in allowable concrete stress is the added strength given to the concrete by the confining action of the steel.

### TABLE 1810.3.2.6

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression&lt;sup&gt;a&lt;/sup&gt;</td>
<td>$0.4 f'_c$</td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>$0.33 f'_c$</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>$0.3 f'_c$</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>$0.33 f'_c$</td>
</tr>
<tr>
<td>Precast non prestressed</td>
<td>$0.33 f'<em>c - 0.27 f</em>{pc}$</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>$0.33 f'<em>c - 0.27 f</em>{pc}$</td>
</tr>
<tr>
<td>2. Non prestressed reinforcement in compression</td>
<td>$0.4 f_y \leq 30,000$ psi</td>
</tr>
<tr>
<td>3. Structural steel in compression</td>
<td>$0.5 f_y \leq 32,000$ psi</td>
</tr>
<tr>
<td>Cores within concrete-filled pipes or tubes</td>
<td>$0.5 f_y \leq 32,000$ psi</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>$0.4 f_y \leq 32,000$ psi</td>
</tr>
<tr>
<td>Pipes or tubes for micro piles</td>
<td>$0.35 f_y \leq 16,000$ psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>$0.6 f_y \leq 0.5 f_y$</td>
</tr>
<tr>
<td>Helical piles</td>
<td>$0.6 f_y \leq 0.5 f_y$</td>
</tr>
<tr>
<td>4. Non prestressed reinforcement in tension</td>
<td>$0.6 f_y$</td>
</tr>
<tr>
<td>Within micropiles</td>
<td>$0.5 f_y \leq 24,000$ psi</td>
</tr>
<tr>
<td>Other conditions</td>
<td>$0.5 f_y \leq 24,000$ psi</td>
</tr>
<tr>
<td>5. Structural steel in tension</td>
<td>$0.5 f_y \leq 32,000$ psi</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>$0.5 f_y \leq 32,000$ psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>$0.35 f_y \leq 16,000$ psi</td>
</tr>
<tr>
<td>Helical piles</td>
<td>$0.6 f_y \leq 0.5 f_y$</td>
</tr>
<tr>
<td>6. Timber</td>
<td>In accordance with the AF&amp;PA NDS</td>
</tr>
</tbody>
</table>

---

a. $f'_c$ is the specified compressive strength of the concrete or grout; $f_y$ is the compressive stress on the gross concrete section due to effective prestress forces only; $f_y$ is the specified yield strength of reinforcement; $f'_c$ is the specified minimum yield stress of structural steel; $f_y$ is the specified minimum tensile stress of structural steel.

b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

---

2012 INTERNATIONAL BUILDING CODE® COMMENTARY 18-53
casing. The general formula for increased allowable stress caused by confinement is:

\[ f_c \cdot 0.33 \left( \frac{1 + 7.5tf_c}{DF_c} \right) \]

where:

- \( f_c \) = Allowable concrete stress.
- \( f'_{cc} \) = Specified concrete strength.
- \( t \) = Thickness of steel shell.
- \( f_y \) = Yield strength of steel.
- \( D \) = Diameter of steel shell.

This formula is from the Portland Cement Association's (PCA) Report on Allowable Stresses in Concrete Piles.

When values for the various terms required by Items 3 through 6 are inserted in the given formula, the resulting allowable stress is 0.40\( f'_{cc} \). Higher allowable stresses would result if, for example, the shell thickness was increased or the shell diameter was decreased. This increased allowable stress caused by confinement applies only to nonaxial load-bearing steel where the stress in the steel is taken in hoop tension instead of axial compression.

Steel pile shells are to be No. 14 gage (U.S. standard) or thicker, but are not to be considered in the design of the pile to carry a portion of the pile load. The equivalent thickness for No. 14 gage material is approximately 0.068 inches (1.7 mm).

The shell for this type of pile must be seamless or have spirally welded seams and be of the strength and configuration required to provide structural confinement of the concrete fill. "Confinement" is the technical qualification that permits the use of increased allowable compressive stresses. Simply stated, the pile casing restrains the concrete in directions perpendicular to the applied stresses.

Item 5 requires that the ratio of the yield strength \( f_y \) of the steel used in pile casings to the design compressive strength of concrete \( f'_{cc} \) is not to be less than six. The yield strength of the steel used for pile casings of the type specified in this section is normally 30,000 psi (207 MPa) or greater. For example, in selecting a casing with a yield strength \( f_y \) of 30,000 psi (207 MPa) and a concrete compressive strength of 3,000 psi (21 MPa), the resulting ratio \( f_y/f'_{cc} \) would be 10, which is greater than the minimum required ratio of six; therefore, the strengths of the pile materials are acceptable. For comparison, use the same steel casing and a specified concrete compressive strength \( f'_{cc} \) of 5,000 psi (34 MPa). The resulting ratio would be exactly six. It can readily be seen that for concrete strengths greater than 5,000 psi (34 MPa), the type of steel used for the casing material would need to yield strengths greater than 30,000 psi (207 MPa).

For example, in order to meet the minimum ratio of six as required by this section, if the concrete compressive strength \( f'_{cc} \) was specified at 6,000 psi (41 MPa), the material to be used for the casing would require a steel yield strength of at least 36,000 psi (248 MPa) conforming to ASTM A 36.

Item 6 limits the nominal diameter of the element to 16 inches (406 mm) in order to qualify for an increase in the allowable design compressive stress.

1810.3.2.8 Justification of higher allowable stresses. Use of allowable stresses greater than those specified in Section 1810.3.2.6 shall be permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A geotechnical investigation in accordance with Section 1803; and
2. Load tests in accordance with Section 1810.3.3.1.2, regardless of the load supported by the element.

The design and installation of the deep foundation elements shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations who shall submit a report to the building official stating that the elements as installed satisfy the design criteria.

In other parts of this chapter, limitations are specified for the stress values used for design purposes. These allowable stresses are stated as a percentage of some limiting strength property of the element's material. For example, allowable design stresses for deep foundation elements made of steel are stated as a percentage of the yield strengths of the several grades of steel typically used for pile construction. For concrete, the allowable design stress is prescribed as a percentage of the specified compression strength. The allowable design stresses permitted for timber elements are based on the natural strengths of the different species of wood used for deep foundations. The values have been developed and tabulated by AF&PA and include reductions in element strengths because of preservative treatment. The allowable design stresses stipulated in the code for the different types of elements provide an adequate factor of safety against the dynamic forces of pile driving that may cause damage to the elements and prevent over-stresses because of loading and subsoil conditions.

This section allows the use of higher allowable stresses when evidence supporting the values is submitted and approved by the building official. The data submitted to the building official should include analytical evaluations and findings from a foundation investigation as specified in Section 1803.5.5, and the results of load tests performed in accordance with the requirements of Section 1810.3.3.1.2. The technical data and the recommendation for the use of higher stress values must come from a registered engineer who is knowledgeable in soil mechanics.
and experienced in the design of pile foundations. This engineer is to supervise the deep foundation design work and witness the installation of the deep foundation so as to certify to the building official that the construction satisfies the design criteria. In any case, the use of greater design stresses is not to result in permitting design loads that are larger than one-half of the test loads (see Section 1810.3.3.1.2).

1810.3.3 Determination of allowable loads. The allowable axial and lateral loads on deep foundation elements shall be determined by an approved formula, load tests or method of analysis.

- There are two general considerations for determining capacity as required for the design and installation of deep foundations. The first consideration involves the determination of the underlying soil or rock characteristics. The second is the application of approved driving formulas, load tests or accepted methods of analysis to determine the element capacities required to resist the axial and lateral loads they will be subjected to, as well as to provide the basis for the proper selection of driving equipment.

1810.3.3.1 Allowable axial load. The allowable axial load on a deep foundation element shall be determined in accordance with Sections 1810.3.3.1.1 through 1810.3.3.1.9.

- This section states the criteria for determining the capacity of deep foundation elements.

1810.3.3.1.1 Driving criteria. The allowable compressive load on any driven deep foundation element where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate driveability for both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1810.3.3.1.2. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven elements. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

- It has been accepted practice for many decades to predict the load capacity of an element by its resistance to driving as determined by a pile-driving formula. The simple premise upon which a pile-driving formula is founded is that as the resistance of a pile to driving increases, the pile’s capacity to support loads also increases. While several pile formulas have been developed over the years, none have been completely dependable. The Engineering News formula is the simplest and probably the most widely used in the United States. This calculation method, as well as other formulas in common use today, have generally shown poor correlations with load test results. However, the comparative differences between pile capacities as determined by driving formulas and the results of load tests are much smaller for soils consisting of free-draining, coarse-grained materials, such as sand and gravel, than for soils consisting of silt, clay or fine, dense sand.

The use of pile-driving formulas to determine pile capacities should generally be avoided, except on small jobs where the piles are to be driven in well-drained granular soils, and the cost of load testing cannot be justified.

1810.3.3.1.2 Load tests. Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6, where the design load for any deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test elements shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one element shall be load tested in each area of uniform subsoil conditions. Where required by the building official, additional elements shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test element as assessed by one of the published methods listed in Section 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section 1810.2.3. In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where such elements are of the same type, size and relative length as the test element; are installed using the same or comparable methods and equipment as the test element; are installed in similar subsoil conditions as the test element; and, for driven elements, where the rate of penetration (e.g., net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.

- The most reliable method for determining pile capacity is by a load test. A load test should be conducted wherever feasible and used where the pile capacity is intended to exceed 40 tons (356 kN) per pile (see Section 1810.3.3.1.1). Test piles are to be of the same type and size intended for use in the permanent foundation and installed with the same equipment, by the same procedure and in the same soils intended or specified for the work. Load tests are to be conducted in accordance with the requirements of ASTM D 1143 or D 4945, which covers procedures for testing vertical or batter foundation piles, individually or in groups, to determine the ultimate pile load (pile capacity) and whether the pile or pile group is capable of supporting the load(s) without excessive or continuous settlement. Recognition, however, must be given to the fact that load-settlement characteristics and pile capacity determinations are based on data derived at the time and under conditions of...
the test. The long-term performance of a pile or group of piles supporting actual loads may produce behaviors that are different than those indicated by load test results. Judgement based on experience must be used to predict pile capacity and expected behavior.

The load-bearing capacity of all piles, except those seated on rock, does not reach the ultimate load until after a period of rest. The results of load tests cannot be deemed accurate or reliable unless there is an allowance for a period of adjustment. For piles driven in permeable soils, such as coarse-grained sand and gravel, the waiting period may be as little as two or three days. For test piles driven in silt, clay or fine sand, the waiting period may be 30 days or longer. The waiting period may be determined by testing (i.e. by redriving piles) or from previous experience.

This section requires that at least one pile be tested in each area of uniform subsoil conditions. The statement should not be misconstrued to mean that the tested area is to have only one uniform stratum of subsurface material, but rather that the soil profile, which may consist of several layers (strata) of different materials, must represent a substantially unchanging cross section in each area to be tested.

The allowable pile load to be used for design purposes is not to be more than one-half of the test pile’s ultimate axial load capacity, as determined in Section 1810.3.3.1.3. In establishing the pile capacity, the registered design professional must consider the tolerable settlement that can be structure dependent (see Section 1810.2.3).

The rate of penetration of production piles must be equal to or less than that of the test pile(s). All production piles should be of the same type, size and approximate length as the prototype test pile, as well as installed with comparable equipment and methods. Driven pile capacities are only valid when the same hammer is used, because different hammers of the same model, though comparable, can actually have different efficiencies. Production piles should also be installed in soils similar to those for the test pile.

1810.3.3.1.3 Load test evaluation methods. It shall be permitted to evaluate load tests of deep foundation elements using any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90% Criterion.
4. Other methods approved by the building official.

This section lists generally accepted methods that can be used to determine the ultimate axial load capacity of test elements. Since no single method applies to all situations that are encountered, this list provides the necessary latitude to select a method of load test evaluation that is appropriate for the type of element being tested, the test procedure and the subsurface conditions. The Davisson Offset Limit is perhaps the most widely used method of test load evaluation. It has proven to provide capacities that are conservative, yet reasonable. The Brinch-Hansen 90% Criterion and Butler-Hoy Criterion are considered a little less conservative than Davisson, but are viable methods of evaluating test elements.

1810.3.3.1.4 Allowable frictional resistance. The assumed frictional resistance developed by any uncased cast-in-place deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1806.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official on the basis of a geotechnical investigation as specified in Section 1803 or a greater value is substantiated by a load test in accordance with Section 1810.3.3.1.2. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless determined by a geotechnical investigation in accordance with Section 1803.

Under certain circumstances, such as when a deep foundation element extends through cohesive soils, like clays, to a bearing stratum of compact sand and gravels, both skin friction and end bearing act together to support the pile. However, the nature of load sharing between the two and whether in fact both act simultaneously can be determined only by a soils investigation. Thus, the code requires that in order to allow the design to be based on both skin friction and end bearing acting simultaneously, the assumption must be justified by a geotechnical investigation.

1810.3.3.1.5 Uplift capacity of a single deep foundation element. Where required by the design, the uplift capacity of a single deep foundation element shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1810.3.3.1.2, using the results of load tests conducted in accordance with ASTM D 3689, divided by a factor of safety of two.

Exception: Where uplift is due to wind or seismic loading, the minimum factor of safety shall be two where capacity is determined by an analysis and one and one-half where capacity is determined by load tests.

Deep foundation elements subjected to uplift forces act in tension and are actually friction piles. The amount of tension that can be sustained by an element depends on the strength of the element material and the frictional or cohesive properties of the soil. Tensile resistance is not necessarily correlated with the bearing capacity of a deep foundation element under compressive load. For example, the tensile resistance of a friction pile in clay will usually be about the same value as its bearing capacity because the skin friction developed in cohesive soils is very large. In comparison, a friction pile installed in granular materials (noncohesive), such as sand, will develop a tensile resistance that is considerably less than its bearing capacity.
Analytical methods can be used to determine the ultimate uplift resistance of a deep foundation element, provided that the properties of the soil are well known. When the ultimate uplift resistance is established by analysis, a safety factor of three must be applied to determine the allowable uplift load of the element.

The response of a vertical or battered element to an axially applied uplift force is best determined by an extraction test performed in accordance with the requirements of ASTM D 3689 and in accordance with the provisions of this section.

Deep foundation elements must be well anchored into the pile cap by adequate connection devices in order to be effective in resisting uplift forces. In turn, the pile cap must also be designed to resist uplift stresses. Sometimes, it is necessary to give special consideration in the design of the element itself to take the tensile stresses imposed by uplift conditions. For example, a cast-in-place or precast concrete element must be designed so that the tensile reinforcement will extend the full length of the element. Special consideration should also be given to the design of splices that must resist tension.

The exception reduces the required factor of safety for uplift due to wind or seismic loading. This is analogous to the long-standing practice of allowing stress increases for earthquake and wind loads.

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be calculated by an approved method of analysis where the deep foundation elements in the group are placed at a center-to-center spacing of at least 2.5 times the least horizontal dimension of the largest single element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual uplift working load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block.

There are two limitations on the capacity of grouped deep foundation elements and the lesser of the two is taken as the allowable uplift. The first limitation is the single-element capacity as determined in the previous section multiplied by the number of elements in the group. The second of these is limited by the weight of the group plus the weight of the soil within the perimeter of the group, in addition to the soil’s shear resistance that will be developed during an uplift loading event.

1810.3.3.1.7 Load-bearing capacity. Deep foundation elements shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

The bearing capacity of a deep foundation element, whether it is a single acting element or part of a group, is determined as a deep foundation soil system. In this respect, element bearing capacity is a function of either the strength properties of the deep foundation element or the supporting strength of the soil. Obviously, the bearing capacity is controlled by the smaller value obtained in the two considerations.

In most cases, the supporting strength of the soil governs the bearing capacity of a deep foundation element. This section requires that the bearing capacity of an individual element or group of elements must not be more than one-half of the ultimate load capacities of the elements as a function of the bearing capacity of the soil.

Sometimes, soils investigations show that weaker layers of soil underlie the intended bearing strata. To avoid damaging settlements, the weaker soils must have a safety factor of 2 or more as determined by analytical methods. Where the safety factor is less than 2, elements must either be driven to deeper bearing soils to obtain adequate and safe support or the design capacity of the elements must be reduced, thus increasing the total number of elements in the foundation system.

1810.3.3.1.8 Bent deep foundation elements. The load-bearing capacity of deep foundation elements discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative element.

This section requires that deep foundation elements that are discovered to have sharp or sweeping bends, usually occurring because of obstructions encountered during driving operations, must be analyzed by an approved method or load tested by a representative element to determine their load-carrying capacity. Where acceptable, such deep foundation elements may be used at a reduced capacity; otherwise, they should be abandoned and replaced.

1810.3.3.1.9 Helical piles. The allowable axial design load, $P_a$, of helical piles shall be determined as follows:

$$P_a = 0.5 P_o$$

(Equation 18-4)

where $P_o$ is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Ultimate capacity determined from load tests.
4. Ultimate axial capacity of pile shaft.
5. Ultimate axial capacity of pile shaft couplings.
SOILS AND FOUNDATIONS

6. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.

- The allowable load on a helical pile is limited to one-half of the ultimate load. The ultimate load is taken as the least of the six criteria that are listed. Helical pile systems having ICC Evaluation Service reports can be viewed at www.icc-es.org (also see ICC Evaluation Service Acceptance Criteria 358).

1810.3.3.2 Allowable lateral load. Where required by the design, the lateral load capacity of a single deep foundation element or a group thereof shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of the load that produces a gross lateral movement of 1 inch (25 mm) at the lower of the top of foundation element and the ground surface, unless it can be shown that the predicted lateral movement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity.

- Because of wind loads, unbalanced building loads, earth pressures and the like, it is inevitable that piers, individual elements or groups of elements supporting buildings or other structures will be subjected to lateral forces. The distribution of these lateral forces to deep foundation elements largely depends on how the loads are carried down through the structural framing system and transferred through the supporting foundation to the deep foundation elements. The amount of lateral load that can be taken by the deep foundation is a function of the type of element used; the soil characteristics, particularly in the upper 10 feet (3048 mm) of the elements; the embedment of the element head (fixity); the magnitude of the axial compressive load on the deep foundation element; the nature of the lateral forces and the amount of horizontal element movement deemed acceptable.

The degree of fixity of the deep foundation element head is an important design consideration under very high lateral loading unless some other method, such as the use of batter piles, is employed to resist lateral loads. The fixing of the deep foundation element head against rotation reduces the lateral deflection. In general, pile butts are embedded 3 inches (76 mm) to 4 inches (102 mm) into the pile cap (see Section 1810.3.11) with no ties to the cap. These pile heads are neither fixed nor free, but somewhere in the middle. Such construction is satisfactory for most loading conditions.

The magnitude of friction developed between the surfaces of two structural elements in contact with each other is a function of the weight of loads applied. The larger the weight, the greater the frictional resistance developed. In the design of pile foundations, frictional resistance between the soil and the bottom of the pile caps (footings) should not be relied on to provide lateral restraint, since the vertical loads are transmitted through the elements to the supporting soil below and not to the ground immediately under the pile caps. Only the weights of the pile caps can supply some frictional resistance because such footings are constructed by placing fresh concrete on the soil, thus providing a positive contact. The weights of the pile caps in comparison to the magnitude of loads and lateral forces transmitted to the piles is nominal, however, and not too significant from a structural design standpoint. Also, in rare occurrences, soil has been known to settle under pile caps, leaving open spaces and thus eliminating the development of any frictional restraint.

Generally, about 1/2-inch (6.4 mm) horizontal movement of a deep foundation element is considered acceptable without tests. Deep foundation elements with their upper sections embedded in deep strata of very soft or soft clays and silts should not be relied on to resist lateral forces of more than 1,000 pounds (4.45 kN) per element.

Where vertical elements are subjected to lateral forces exceeding acceptable limitations, the use of batter piles may be required. Lateral forces on many structures are also resisted by the embedded foundation walls and the sides of the pile caps.

The allowable lateral load capacity of a pier, single element or group of elements is to be determined either by approved analytical methods or load tests. Load tests are to be conducted to produce lateral forces that are twice the proposed design load; however, in no case is the allowable deep foundation element load to exceed one-half of the test load, which produces a gross lateral deep foundation element movement of 1 inch (25 mm) as measured at the ground surface.

1810.3.4 Subsiding soils. Where deep foundation elements are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the elements by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the element, the allowable stresses specified in this chapter shall be permitted to be increased where satisfactory substantiating data are submitted.

- Where compacted fill or other surcharge loads are placed over compressible soils, the underlying material will consolidate because of the added load. The depth to suitable bearing material will, over a period of time, be shifted downward by the forces of the subsiding soil. Such forces caused by the weight of the fill are transmitted to the elements by skin friction and, in effect, serve as added loads on the elements. The magnitude of such loads must be determined by accepted analytical methods and be taken into account in the design of foundations.

1810.3.5 Dimensions of deep foundation elements. The dimensions of deep foundation elements shall be in accor-
dance with Sections 1810.3.5.1 through 1810.3.5.3, as applicable.

- This section groups together any dimensional limitations that apply to various types of elements.

**1810.3.5.1 Precast.** The minimum lateral dimension of precast concrete deep foundation elements shall be 8 inches (203 mm). Corners of square elements shall be chamfered.

- This section prescribes the minimum dimension for precast concrete elements based on the size required to withstand the driving operation. Chamfered corners consist of rounding off or smoothing the corners of a square pile. The triangular portions of the corners are typically weak spots and chamfering them reduces the risk of concrete spalling, cracking or breakage during driving.

**1810.3.5.2 Cast-in-place or grouted-in-place.** Cast-in-place and grouted-in-place deep foundation elements shall satisfy the requirements of this section.

- Dimensional limitations applicable to deep foundation elements that are cast-in-place are dependent on whether the element is cased or uncased.

**1810.3.5.2.1 Cased.** Cast-in-place deep foundation elements with a permanent casing shall have a nominal outside diameter of not less than 8 inches (203 mm).

- Steel-cased piles are the most widely used type of cast-in-place concrete deep foundation element. Essentially, they consist of mandrel-driven, light-gage steel shells or thin-walled pipes that are left permanently in place, reinforced as required by the design and filled with concrete. The shell is either a constant section for the full length of the element or a step-tapered shape. The presence of the casing permits a higher allowable stress in the concrete than for an uncased pile (see Table 1810.3.2.6).

**1810.3.5.2.2 Uncased.** Cast-in-place deep foundation elements without a permanent casing shall have a diameter of not less than 12 inches (305 mm). The element length shall not exceed 30 times the average diameter.

Exception: The length of the element is permitted to exceed 30 times the diameter, provided the design and installation of the deep foundations are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and deep foundations. The registered design professional shall submit a report to the building official stating that the elements were installed in compliance with the approved construction documents.

- The dimensional relationship between the diameter and length of a deep foundation element has been utilized for many decades and is based on the premise that an element under axial load behaves somewhat as a column that could be susceptible to buckling. On the other hand, it has also been established through technological advancements, research and experience that deep foundation elements embedded in soils, even in soft materials, do not behave as free-standing columns and the risk of buckling is extremely low. Notwithstanding this kind of evidence, the code limitations placed on dimensional requirements based on diameter-to-length ratios have become accepted practices.

**1810.3.5.2.3 Micropiles.** Micropiles shall have an outside diameter of 12 inches (305 mm) or less. The minimum diameter set forth elsewhere in Section 1810.3.5 shall not apply to micropiles.

- The micropile does not have a minimum required size—only the maximum diameter is specified.

**1810.3.5.3 Steel.** Steel deep foundation elements shall satisfy the requirements of this section.

- These dimensional limits are established for steel H-piles, pipes and tubes.

**1810.3.5.3.1 H-piles.** Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of 1/4 inch (9.5 mm).

- Dimensional requirements for manufacturing structural steel H-piles are provided in this section. H-piles are proportioned to withstand the large impact stresses imposed on piles during hard driving. The thicknesses of the flanges and the web of a rolled-steel H-pile section are made equal in order to avoid damage that could occur during hard driving if the piles were proportioned with a mixture of thick and thinner parts. Flange widths are proportioned in relation to the depth of the pile section to provide rigidity in the weak axis.

It would seem unnecessary to repeat the dimensioning requirements for H-piles in the code, since they were originally created by the steel industry and have for decades been the industry standard used in the manufacture of hot-rolled steel shapes. The main purpose is to provide the dimensional basis for the fabrication of similar pile products made principally of welded steel plates and other necessary steel parts.

While it is the general preference and practice to use rolled-steel piles, occasionally it becomes necessary to fabricate special pile sections because of time problems imposed by mill scheduling and delivery; a special need for heavier H-pile sections than are customarily available; an immediate need for replacement piles; or for any other reason. The dimensional requirements contained in this section regulate the fabrication of such special piles and provide the building official with a basis for approval. Fabricated pile materials are to comply with the requirements of Section 1810.3.2.3.

This section requires that the flange projection not exceed 14 times the minimum thickness of metal in
either the flange or the web. The measurement of the flange projection is shown in Figure 1810.3.5.3.1 and can be calculated by the indicated formula:

\[ P = \frac{W - t_f}{2} \text{ or } \frac{W - t_w}{2} \]

where \( W \) is the flange width, \( t_w \) is the web thickness, \( t_f \) is the flange thickness and \( P \) is the flange projection.

For example, the dimensions of an HP 14 x 73 pile section are as follows:

The width of flange, \( W = 14.585 \) inches.
The thickness of the flange, \( t_f = 0.505 \) inch.
The thickness of the web, \( t_w = .505 \) inch.
Therefore, the actual flange projection is:

\[ P = \frac{14.585 - .505}{2} = 7.04 \text{ inches (179 mm)} \]

The maximum allowable flange projection is \( 14 \times t_f \) or \( 14 \times t_w \) (whichever is smaller).

Allowable flange projection: \( 14 \times 0.505 = 7.07 \) inches (179.5 mm), which is greater than the actual flange projection of 7.05 inches (179.8 mm); therefore, an HP 14 x 73 pile is acceptable.

This section also specifies that flange widths are not to be less than 80 percent of the depth of the pile section. In actuality, practically all H-piles are manufactured so that their flanges are slightly greater in width (by fractions of an inch) than the depths of the sections. In fact, piles are squared so that their flange widths are about equal to their depth.

Flange and web thickness is not to be less than \( \frac{3}{8} \) inch (9.5 mm). In H-piles, the flange thicknesses are equal to the web thickness.

1810.3.5.3.2 Steel pipes and tubes. Steel pipes and tubes used as deep foundation elements shall have a nominal outside diameter of not less than 8 inches (203 mm). Where steel pipes or tubes are driven open ended, they shall have a minimum of 0.34 square inch (219 mm²) of steel in cross section to resist each 1,000 foot-pounds (1356 Nm) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where a pipe or tube with wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided. Concrete-filled steel pipes or tubes in structures assigned to Seismic Design Category C, D, E or F shall have a wall thickness of not less than \( \frac{3}{16} \) inch (5 mm). The pipe or tube casing for socketed drilled shafts shall have a nominal outside diameter of not less than 18 inches (457 mm), a wall thickness of not less than \( \frac{3}{8} \) inch (9.5 mm) and a suitable steel driving shoe welded to the bottom; the diameter of the rock socket shall be approximately equal to the inside diameter of the casing.

Exceptions:

1. There is no minimum diameter for steel pipes or tubes used in micropiles.
2. For mandrel-driven pipes or tubes, the minimum wall thickness shall be \( \frac{1}{16} \) inch (2.5 mm).

Because of their uniform cross section from butt to tip, steel pipe elements provide unvarying resistance to bending and lateral forces applied in any direction. This section requires that seamless or welded-steel pipe and tube elements have a nominal outside diameter of no less than 8 inches (203 mm). The 8-inch (203 mm) minimum diameter is necessary in order to inspect the inside of the pile to observe any damage that may have occurred during the driving process. Smaller diameter pipes could allow soil to bridge the sides and cause plugging at their tips. The minimum diameter also provides stiffness for driving purposes.

A pipe to be driven open ended must be a minimum cross-sectional area which can be determined by one of two methods. The first is a long-standing rule of thumb that relates pile area to the hammer energy. The second approach recognizes the more up-to-date wave equation. The latter approach is commonly used to evaluate the induced driving stresses in piles.

Exception 1 clarifies that the minimum diameter is not applicable to micropiles. Exception 2 permits a thinner pipe thickness when it is mandrel-driven.

1810.3.5.3.3 Helical piles. Dimensions of the central shaft and the number, size and thickness of helical bearing plates shall be sufficient to support the design loads.

The dimensions of the helical pile component are not explicitly limited. Rather the code merely requires that the dimensions are established by the required capacity. Helical pile systems having ICC Evaluation Service reports can be viewed at www.icc-es.org
(also see ICC Evaluation Service Acceptance Criteria
358).

1810.3.6 Splices. Splices shall be constructed so as to provide
and maintain true alignment and position of the component
parts of the deep foundation element during installation and
subsequent thereto and shall be designed to resist the axial
and shear forces and moments occurring at the location of the
splice during driving and for design load combinations.
Where deep foundation elements of the same type are being
spliced, splices shall develop not less than 50 percent of the
bending strength of the weaker section. Where deep founda-
tion elements of different materials or different types are
being spliced, splices shall develop the full compressive
strength and not less than 50 percent of the tension and bend-
ing strength of the weaker section. Where structural steel
cores are to be spliced, the ends shall be milled or ground to
provide full contact and shall be full-depth welded.

Splices occurring in the upper 10 feet (3048 mm) of the
embedded portion of an element shall be designed to resist at
allowable stresses the moment and shear that would result
from an assumed eccentricity of the axial load of 3 inches (76
mm), or the element shall be braced in accordance with Sec-
tion 1810.2.2 to other deep foundation elements that do not
have splices in the upper 10 feet (3048 mm) of embedment.

While it is physically and economically better to drive
deep foundation elements in one piece, site condi-
tions sometimes necessitate that elements be driven in
spliced sections. For example, when the soil or rock-
-bearing stratum is located so deep below the ground that the leads on the driving equipment will
not receive full-length elements, it becomes neces-
sary to install the elements sectionally or, where pos-
sible, to take up the extra length by setting the tip in a
preexcavated hole (see commentary, Section
1810.4.4). When elements are installed in areas such
as existing buildings with restricted headroom, they
are also required to be placed in spliced sections.
There are a number of other reasons for field-splicing
elements, such as restrictions on shipping lengths,
the use of composite elements, etc.

This provision requires that splices be constructed
so as to provide and maintain true alignment and
position of the element sections during installation.
Furthermore, splices must be of sufficient strength to
transmit safely the vertical and lateral loads on the
elements, as well as to resist the bending stresses
that may occur at splice locations during the driving
operations and under long-term service loads. Where
the sections being spliced are the same type of ele-
ment, splices are to develop at least 50 percent of the
bending strength of the weaker section.

There are different methods employed in splicing
elements based on the different materials used in pile
construction. For example, timber piles are spliced by
one of two commonly used methods. The first method
uses a pipe sleeve with a length of about four to five
times the diameter of the pile. The butting ends of the
pile are sawn square for full contact of the two pile
sections, and the spliced portions of the timber pile
are trimmed smoothly around their periphery so as to
fit tightly into the pipe sleeve. The other splicing
method involves the use of steel straps and bolts.
The butting ends of the pile sections are sawn square
for full contact and proper alignment and the four
sides are planed flat to receive the splicing straps.
This type of splicing can resist some uplift forces.

Splicing of precast concrete piles usually occurs at
the head portions of the piles where, after the piles are
driven to their required depth, pile heads are cut
off or spliced to the desired elevation for proper
embedment in the concrete pile caps. Any portion of
the pile that is cracked or shattered caused by the
driving operations or cutting off of pile heads should
be removed and spliced with fresh concrete. To cut
off a precast concrete pile section, a deep groove is
chiseled around the pile exposing the reinforcing
bars, which are then cut off (by torch) to desired
heights or extensions. The pile section above the
groove is snapped off (by crane) and a new pile sec-
tion is freshly cast to tie in with the precast pile.

Steel H-piles are spliced in the same manner as
steel columns, normally by welding the sections
together. Welded splices may be welded-plate or bar
splices, butt-welded splices, special welded splice fit-
tings or a combination of these. Spliced materials
should be kept on the inner faces of the H-pile sec-
tions to avoid forcing a hole in the ground larger than
the pile, causing at least a temporary loss in frictional
value and lateral support that might result in exces-
sive bending stresses.

Steel pipe piles may be spliced by butt welding,
sometimes using straps to guide the sections and
provide more strength to the welded joint. Another
method is to use inside sleeves having a driving fit,
with a flange extending between the pipe sections. By
applying bituminous cement or compound on the out-
side of the ring before driving, a water-tight joint is
obtained.

The strength of a composite pile is governed not
only by the weaker section, but also by the strength
and details of the splice that joins and holds the pile
sections together.

Guidance is also provided for splicing sections of
different element types—i.e., a composite element.
There are several problems associated with the splic-
ing of sections comprising a composite pile. For
example, if the length of a composite pile is of such
dimension that it will fit in the leaders of the driving
equipment, then the full length of the spliced pile can
be driven as a continuous operation. However, if the
pile is too long to fit in the leaders as a single unit, the
pile must be installed in sections, and the driving
would have to be interrupted in order to make the
splice in the leaders.

It is most important that splicing devices be made
in such a way that their installation will be simple and
quick. When pile driving must be stopped to make a
splice in the leaders, the time expended should not be of a duration that would allow the soil to set up (soil freeze) so that continued driving would produce excessive stresses in the pile.

Sometimes when the predetermined length of a composite pile is too long to fit in the leaders, but not overly long, the difference in length between the pile and the leaders may be made up by inserting the lower end of the spliced unit in a preexcavated hole, thus allowing for continuous driving of the pile (see Section 1810.4.4).

Another problem related to splicing occurs in trying to accurately align the pile sections. When the lower section of a composite pile is driven nearly full length into the ground, it is very difficult to determine its true direction, particularly if the pile has drifted off line because of soil pressures, subsurface obstructions, improper driving or for any other reason. When a pile section is connected to a section already driven in the ground, the upper section should be installed in the same direction as the longitudinal axis of the lower section, even though the direction may not be vertical. It is better to maintain a misdirection rather than to try to correct a situation and create a bend at the pile joint.

The selection of the splicing method to be used for assembling a composite pile should be based on the driving and service load stresses that must be resisted by the pile. Splices should be designed to prevent separation of the pile sections during construction and thereafter. Special consideration must be given to the design of splices to resist uplift forces, whether the piles are subject to heaving or specifically designed as tension piles. Designing the splice strong enough to develop about one and one-half to two times the design uplift force is proper engineering practice. Pile splices must also be designed to resist compressive, bending and shear stresses imposed by construction and service loads.

Additionally, splices that occur in the upper 10 feet (3048 mm) of element embedment are to be designed to resist the bending moments and shears at the allowable stress levels of the element material, based on an assumed element load eccentricity of 3 inches (76 mm), unless the element is properly braced. Proper bracing of a spliced element is deemed to exist if stability of the element group is present in accordance with Section 1810.2.2, provided that the other elements in the group do not have splices in the upper 10 feet (3048 mm) of their embedded length.

1810.3.6.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F splices of deep foundation elements shall develop the lesser of the following:

1. The nominal strength of the deep foundation element; and

2. The axial and shear forces and moments from the seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

- More stringent minimum strength requirements are stated in this section that apply to structures with moderate to high seismic risk.

1810.3.7 Top of element detailing at cutoffs. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of a deep foundation element, provisions shall be made so that those specified lengths or extents are maintained after cutoff.

- This section accounts for the condition where an element encounters refusal at a shallower depth than intended and an unanticipated portion of the element is cut off. It is imperative that the required reinforcement be provided at the top of the element even when excess length is cut off.

1810.3.8 Precast concrete piles. Precast concrete piles shall be designed and detailed in accordance with Sections 1810.3.8.1 through 1810.3.8.3.

- Precast concrete piles are manufactured as nonprestressed (conventionally reinforced) or prestressed. Both types can be formed by bedcasting, spinning (centrifugal casting), vertical casting, slip forming or extrusion methods. They are usually made in square, octagonal or round shapes (see Figure 1810.3.8). Precast piles are manufactured as solid units or may be made with a hollow core. They can also be made with internal jet pipes or inspection ducts.

Precast piles are generally ordered in predetermined lengths based on the findings and analysis of soil exploratory work conducted at the project site. This type of element is a displacement pile that is normally installed with pile-driving equipment.

Precast concrete piles are to be designed in accordance with ACI 318. Nonprestressed concrete piles are usually considered for lengths of 40 to 50 feet (12 192 to 15 240 mm), while prestressed concrete piles are usually considered for lengths of 60 to 100 feet (18 288 to 30 480 mm). The general loading range for precast concrete piles is 40 to 400 tons (355 to 3558 kN).

Advantages of using precast concrete piles include their high load capacities and corrosion resistance. Disadvantages include their vulnerability to damage associated with handling, high breakage rates, particularly when spliced, and the considerable displacement of soil.

1810.3.8.1 Reinforcement. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced center to center as follows:

- At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then

- At not more than 4 inches (102 mm), for the remainder of the first 2 feet (610 mm) from each end; and then
3. At not more than 6 inches (152 mm) elsewhere. The size of ties and spirals shall be as follows:

1. For piles having a least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).

2. For piles having a least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).

3. For piles having a least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than $\frac{1}{4}$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

For precast piles, the longitudinal steel must be set in a symmetrical arrangement in the pile. To resist high-impact stresses, lateral steel ties used for confining the longitudinal reinforcement are to be provided at each end of the pile for a distance of 2 feet (610 mm) or more and be closely spaced at 4 inches (102 mm) o.c., except that the first five ties from each end are to be spaced at 1-inch (25 mm) centers. Between these two closely tied ends, the longitudinal steel must be similarly tied at spacings not to exceed 6 inches (152 mm) o.c. so as to provide a reinforcing cage that will keep the pile from buckling or cracking during handling.

1810.3.8.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall comply with the requirements of Sections 1810.3.8.2.1 through 1810.3.8.2.3.

Precast nonprestressed (conventionally reinforced) concrete piles are manufactured from concrete and have reinforcement consisting of a steel reinforcing cage made up of several longitudinal bars or tie steel in the form of individual hoops or spirals.

1810.3.8.2.1 Minimum reinforcement. Longitudinal reinforcement shall consist of at least four bars with a minimum longitudinal reinforcement ratio of 0.008.

- This section specifies the minimum amount of longitudinal reinforcement where seismic effects are minimal or low. See Sections 1810.3.8.2.2 and 1810.3.8.2.3 for reinforcement requirements for moderate and high seismic regions.

1810.3.8.2.2 Seismic reinforcement in Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F, precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ratio shall be 0.01 throughout the length. Transverse reinforcement shall consist of closed ties or spirals with a minimum $\frac{3}{4}$ inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar or 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile.

- This section includes moderately ductile detailing requirements for precast nonprestressed piles in buildings assigned to Seismic Design Categories C and higher. The minimum longitudinal and transverse reinforcement requirements are so that piles will be able to accommodate seismically induced ground deformations. The 1-percent minimum longitudinal reinforcement is a standard requirement for rein-

![Figure 1810.3.8](image)

**Figure 1810.3.8**

**PRECAST NONPRESTRESSED AND PRESTRESSED CONCRETE PILES**
forced concrete columns. A 6-bar-diameter or 6-inch (152 mm) spacing of transverse reinforcement is a fairly common requirement to prevent buckling of longitudinal compression reinforcement. The transverse reinforcement spacing requirements of this section for the confinement region of the element adjacent to the pile cap is somewhat less stringent than that, allowing a 9-bar-diameter or 6-inch (152 mm) spacing. Outside of this region, only the 6-inch spacing applies as already required by Section 1810.3.8.1.

1810.3.8.2.3 Additional seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, transverse reinforcement shall be in accordance with Section 1810.3.9.4.2.

- The confinement region of the element requires the more stringent and demanding detailing provided in the referenced sections of ACI 318, which are the requirements for special moment frames. These increased transverse reinforcement requirements are intended to provide a high degree of ductility in the upper portion (confinement region) of the element. Experience has shown that concrete elements tend to hinge or sustain damage immediately below the pile cap; therefore, tie spacing is reduced in this area to better confine the concrete.

1810.3.8.3 Precast prestressed piles. Precast prestressed concrete piles shall comply with the requirements of Sections 1810.3.8.3.1 through 1810.3.8.3.3.

- Precast prestressed concrete piles are either of the pretensioned or post-tensioned type. Pretensioned type of piles are normally cast in a plant in their full lengths, as predetermined by soils investigations and engineering analyses. The reinforcement in the concrete pile consists of tendons (stressing steel) that are tensioned before the concrete is placed. After casting and when the concrete has attained sufficient strength, the stretched tendons are released from their anchorages, thus placing the pile in continuous compression. In comparison, post-tensioned type of piles are made in a plant or on the job site, and the tendons are released after the concrete has hardened. In addition to the tendons, this type of pile also contains mild reinforcing steel to resist the handling stresses before the stressing steel is tensioned.

One of the primary advantages of prestressed over nonprestressed concrete piles is durability. Since the concrete is under continuous compression, hairline cracks are kept tightly closed and, thus, are more durable. Another advantage is that the tensile stresses that can develop in the concrete under certain driving conditions are less critical. Prestressed concrete piles are best suited for friction piles in sand, gravel and clays.

The purpose for prestressing concrete piles is to place the concrete under continuous compression so that any hairline cracks that may develop will be kept tightly closed and to prevent possible injury to the pile from tensile stresses that may occur during installation operations. The handling and driving of prestressed piles do not require the same degree of care as needed to install conventionally reinforced concrete piles. As a general rule, prestressed piles are more durable than precast reinforced piles.

1810.3.8.3.1 Effective prestress. The effective prestress in the pile shall not be less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

- For prestressed concrete piles, the effective prestress (the stress remaining in the pile after all losses have occurred—excluding the effect of superimposed loads and the weight of the pile) is not to be less than the specified values for the various lengths. Experience has shown that an effective prestress less than the minimum value prescribed herein is sometimes inadequate in preventing or controlling cracking of the concrete during the handling and installation operations. This section also requires the effective prestress to be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel and the tensile stresses in the steel not to exceed the values set forth in ACI 318.

1810.3.8.3.2 Seismic reinforcement in Seismic Design Category C. For structures assigned to Seismic Design Category C, precast prestressed piles shall have transverse reinforcement in accordance with this section. The volumetric ratio of spiral reinforcement shall not be less than the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

\[
\rho_s = 0.12 f'_c / f_{ys}
\]

(Equation 18-5)

where:

- \( f'_c \) = Specified compressive strength of concrete, psi (MPa).
- \( f_{ys} \) = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

\( \rho_s \) = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 18-5 shall be provided below the upper 20 feet (6096 mm) of the pile.

- This section gives requirements for precast prestressed piles supporting structures assigned to Seismic Design Category C. The minimum spiral reinforcement requirement that results in ductile prestressed concrete piles is based on Precast/Prestressed Concrete Institute's (PCI) Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling. This document incorporates work done on piles in the United States and New Zealand.

These piles exhibit larger curvatures in the top 20 feet (6096 mm). This section requires (confine...
spiral reinforcement as determined by Equation 18-5. Based on ACI 318, this formula is deemed to provide for moderate ductility. In the lower portion of the element, the required reinforcing is reduced by one-half.

1810.3.8.3.3 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, precast prestressed piles shall have transverse reinforcement in accordance with the following:

1. Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.

2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.

3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand or 8 inches (203 mm), whichever is smallest.

4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of each spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Section 12.14.3 of ACI 318.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

   \[
   \rho_s = 0.25 \left( \frac{f'_{ce}}{f_{sh}} \right) \left( \frac{A_g}{A_{ch}} \right) \left( 0.5 + 1.4P \left( \frac{f'_{ce}}{A_g} \right) \right) 
   \]

   (Equation 18-6)

   but not less than

   \[
   \rho_s = 0.12 \left( \frac{f'_{ce}}{f_{sh}} \right) \left[ 0.5 + 1.4P \left( \frac{f'_{ce}}{A_g} \right) \right] \times 0.12 \frac{f'_{ce}}{f_{sh}} 
   \]

   (Equation 18-7)

   and need not exceed:

   \[
   \rho_s = 0.021 
   \]

   (Equation 18-8)

   where:

   \( A_g \) = Pile cross-sectional area, square inches (mm²).

   \( A_{ch} \) = Core area defined by spiral outside diameter, square inches (mm²).

   \( f'_{ce} \) = Specified compressive strength of concrete, psi (MPa).

   \( f_{sh} \) = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

   \( P \) = Axial load on pile, pounds (kN), as determined from Equations 18-5 and 16-7.

   \( \rho_s \) = Volumetric ratio (vol. spiral/vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. Where transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, \( s \), and perpendicular dimension, \( h_c \), shall conform to:

   \[
   A_{sh} = 0.3s \ h_c \ \left( f'_{ce} / f_{sh} \right) \left( A_g / A_{ch} \right) \left( 0.5 + 1.4P \left( f'_{ce} / A_g \right) \right) 
   \]

   (Equation 18-9)

   but not less than:

   \[
   A_{sh} = 0.12s \ h_c \ \left( f'_{ce} / f_{sh} \right) \left( 0.5 + 1.4P \left( f'_{ce} / A_g \right) \right) 
   \]

   (Equation 18-10)

   where:

   \( f_{sh} \) = yield strength of transverse reinforcement ≤ 70,000 psi (483 MPa).

   \( h_c \) = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

   \( s \) = Spacing of transverse reinforcement measured along length of pile, inch (mm).

   \( A_{sh} \) = Cross-sectional area of transverse reinforce-ment, square inches (mm²).

   \( f'_{ce} \) = Specified compressive strength of concrete, psi (MPa).

   The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

   Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

This section clarifies that Chapter 21 of ACI 318 does not generally apply to precast prestressed concrete elements unless it is explicitly referenced. That ACI 318 chapter was never intended for deep foundation elements.

The provisions found in this section are based on PCI’s Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling. This document incorporates work done on piles in the United States and New Zealand.

The maximum volumetric ratio set forth in Equation 18-8 is based on testing that has shown the 0.021 maximum is sufficient for the smaller square precast prestressed concrete elements to perform in a ductile manner.

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be designed and detailed in accordance with Sections 1810.3.9.1 through 1810.3.9.6.

This section sets forth the design and detailing requirements for cast-in-place concrete deep foundation elements.
1810.3.9.1 Design cracking moment. The design cracking moment \( (\phi M_n) \) for a cast-in-place deep foundation element not enclosed by a structural steel pipe or tube shall be determined using the following equation:

\[
\phi M_n = 3\sqrt{f'c}S_m \quad \text{(Equation 18-11)}
\]

For SI: \( \phi M_n = 0.25\sqrt{f'c}S_m \)

where:

- \( f'c \): Specified compressive strength of concrete or grout, psi (MPa).
- \( S_m \): Elastic section modulus, neglecting reinforcement and casing, cubic inches (mm\(^3\)).

- For both unencased and cased cast-in-place deep foundation elements (but not concrete-filled pipes and tubes), reinforcement must be provided where moments exceed a reasonable lower boundary for the capacity of the plain concrete section. This criterion is consistent with ACI 318.

1810.3.9.2 Required reinforcement. Where subject to uplift or where the required moment strength determined using the load combinations of Section 1605.2 exceeds the design cracking moment determined in accordance with Section 1810.3.9.1, cast-in-place deep foundations not enclosed by a structural steel pipe or tube shall be reinforced.

- This section lists two conditions under which reinforcing must be provided in cast-in-place elements. Note the cracking moment is checked using the strength level load effects.

1810.3.9.3 Placement of reinforcement. Reinforcement where required shall be assembled and tied together and shall be placed in the deep foundation element as a unit before the reinforced portion of the element is filled with concrete.

Exceptions:

1. Steel dowels embedded 5 feet (1524 mm) or less shall be permitted to be placed after concreting, while the concrete is still in a semifluid state.

2. For deep foundation elements installed with a hollow-stem auger, tied reinforcement shall be placed after elements are concreted, while the concrete is still in a semifluid state. Longitudinal reinforcement without lateral ties shall be placed either through the hollow stem of the auger prior to concreting or after concreting, while the concrete is still in a semifluid state.

3. For Group R-3 and U occupancies not exceeding two stories of light-frame construction, reinforcement is permitted to be placed after concreting, while the concrete is still in a semifluid state, and the concrete cover requirement is permitted to be reduced to 2 inches (51 mm), provided the construction method can be demonstrated to the satisfaction of the building official.

Main reinforcement consisting of a cage of longitudinal deformed reinforcing bars tied together with individual steel hoops or a spiral must be placed and securely held in position in the pile opening (cased or uncased) prior to the placement of concrete. Reinforcement required for resisting tensile stresses imposed by uplift forces may consist of a single bar or other structural steel unit or a cluster of bars placed at the center for the full length of the element.

Exception 1 recognizes the typical procedure for constructing cast-in-place concrete elements is to tie the head of the pile with the pile cap, inserting dowels in the freshly placed concrete pile to obtain an embedment of about 5 feet (1524 mm).

Exception 2 recognizes this restriction is not applicable to auger-injected concrete piles. Unlike drilled cast-in-place concrete piles, caged reinforcement for auger-placed piles cannot be installed prior to filling the pile hole with concrete because the hollow-stem auger must be positioned in the hole at all times during drilling and concreting operations. To facilitate this, the cage must be pushed through the concrete in fluid form after the auger has been withdrawn. The problem associated with the placement of caged reinforcement in this way, particularly where it involves long cages, is that it cannot be determined if the assembly has been positioned appropriately within the filled hole such that the reinforcement will have the required minimum cover at all places.

Exception 3 exempts up to two stories of light-frame construction in Group R-3 and U occupancies.

1810.3.9.4 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, reinforcement shall be provided in accordance with Section 1810.3.9.4.1. Where a structure is assigned to Seismic Design Category D, E or F, reinforcement shall be provided in accordance with Section 1810.3.9.4.2.

Exceptions:

1. Isolated deep foundation elements supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where detailed so the element is not subject to lateral loads and the soil provides adequate lateral support in accordance with Section 1810.2.1.

2. Isolated deep foundation elements supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where the lateral load, \( E \), to the top of the element does not exceed 200 pounds (890 N) and the soil provides adequate lateral support in accordance with Section 1810.2.1.

3. Deep foundation elements supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required
by rational analysis but with not less than two No. 4 bars, without ties or spirals, where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations with overload factor in Section 12.4.3.2 or 12.14.3.2 of ASCE 7 and the soil provides adequate lateral support in accordance with Section 1810.2.1.

4. Closed ties or spirals where required by Section 1810.3.9.4.2 shall be permitted to be limited to the top 3 feet (914 mm) of deep foundation elements 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

This section prescribes the necessary seismic reinforcement for cast-in-place deep foundation elements. Exception 1 exempts isolated deep foundation elements from the transverse reinforcement requirements of this section. Exceptions 2 through 4 allow lesser reinforcement than what is required by this section for residential structures, given certain conditions.

1810.3.9.4.1 Seismic reinforcement in Seismic Design Category C. For structures assigned to Seismic Design Category C, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, shall be provided throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. One-third of the element length;
2. A distance of 10 feet (3048 mm);
3. Three times the least element dimension; and
4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

Transverse reinforcement shall consist of closed ties or spirals with a minimum 1/4" (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of 6 inches (152 mm) or 8-longitudinal-bar diameters, within a distance of three times the least element dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 16 longitudinal bar diameters throughout the remainder of the reinforced length.

Exceptions:
1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. A spiral-welded metal casing of a thickness not less than manufacturer’s standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

Longitudinal and transverse reinforcement requirements prescribed by this section result in moderate ductility in buildings assigned to Seismic Design Category C to withstand seismically induced ground deformations that they can encounter. Separate transverse reinforcement is specified for the portions of the element within and outside of the potential plastic hinge zone.

The purpose of this section is to include element bending, which is a result of ground horizontal movement during an earthquake, in the structural design. The reinforcement in the element, required to resist the tension caused by element bending, increases the ductility of the foundation such that bending or shear failure is precluded.

1810.3.9.4.2 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.005, shall be provided throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. One-half of the element length;
2. A distance of 10 feet (3048 mm);
3. Three times the least element dimension; and
4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

Transverse reinforcement shall consist of closed ties or spirals no smaller than No. 3 bars for elements with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger elements. Throughout the remainder of the reinforced length outside the regions with transverse confinement reinforcement, as specified in Section 1810.3.9.4.2.1 or 1810.3.9.4.2.2, the spacing of transverse reinforcement shall not exceed the least of the following:

1. 12 longitudinal bar diameters;
2. One-half the least dimension of the element; and
3. 12 inches (305 mm).

Exceptions:
1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. A spiral-welded metal casing of a thickness not less than manufacturer’s standard gage No. 14 gage
(0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

- This section is similar in intent to Section 1810.3.9.4.1, with increased minimum reinforcement requirements for ductility during earthquake ground motion that may be experienced by buildings assigned to Seismic Design Category D or higher. Separate transverse reinforcement requirements are given for portions of an element within and beyond the potential plastic hinge zone.

- The purpose of this section is to include pile bending, which is a result of ground horizontal movement during an earthquake, in the structural design. The reinforcement in the pile, required to resist tension caused by pile bending, increases the ductility of the foundation such that bending or shear failure is precluded. The shear strength and confining ability of spiral-welded metal casing eliminates the need for special pile ties, which is the basis for Exception 2.

1810.3.9.4.2.1 Site Classes A through D. For Site Class A, B, C or D sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318 within three times the least element dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.6.4.4(a) of ACI 318 shall be permitted.

- The specified confinement reinforcing similar to ACI 318 requirements for special moment frames is reduced in competent soils in recognition of the confinement attributed to those soils.

1810.3.9.4.2.2 Site Classes E and F. For Site Class E or F sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft- to medium-stiff clay.

- The specified confinement reinforcing is similar to ACI 318 requirements for special moment frames.

1810.3.9.5 Belled drilled shafts. Where drilled shafts are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

- Deep foundation elements are sometimes belled out at their bottoms to increase bearing areas. Belled bottoms can be made by either hand excavation or a mechanical method involving underreaming with a belling bucket, as is commonly used in the construction of drilled elements.

- The soil should be sufficiently cohesive so the roof of the bell will not collapse during excavation, cleanout operations and the placement of fresh concrete. Where the character of the soil is such that attempts at belting out the shaft at the bottom might not produce acceptable results, it would be preferable to continue the shaft into soil strata with better load-bearing values, allowing the loads to be carried by the smaller belled bottoms or to such depths that would permit the soils to carry the loads by side friction. The building official should ascertain the suitability of the soil for bell construction through geotechnical investigations and reports and by the recommendations of the design professional.

- Bells must have vertical edges at their bottoms that are equal to the thickness requirements for concrete footings. The purpose of this specification is to prevent shear breaks in angled edges caused by soil pressures. Bell slopes (sides) are not to have sides that are less than 60 degrees (1 rad) from the horizontal, unless the effects of vertical shear are considered in the design. Figure 1810.3.9.5 illustrates these requirements.

1810.3.9.6 Socketed drilled shafts. Socketed drilled shafts shall have a permanent pipe or tube casing that extends down to bedrock and an uncased socket drilled into the bedrock, both filled with concrete. Socketed drilled shafts shall have reinforcement or a structural steel core for the length as indicated by an approved method of analysis.

- The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the element with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe or tube casing. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket.

- Where a structural steel core is used, the gross cross-sectional area of the core shall not exceed 25 percent of the gross area of the drilled shaft.

- The socketed drilled shaft is a high-load capacity, end-bearing type of deep foundation element. Essentially, it is constructed as a cased, cast-in-place concrete element formed by driving a thick-walled, open-ended steel pipe down to suitable rock material, cleaning out the soil materials within the pipe, drilling a socket into the rock, inserting a structural steel core or reinforcement into the pipe and then filling the pipe and drilled socket with concrete. The core material used in the pipe casing is a structural steel shape such as a wide-flange section. Steel rails are also used. Core material is installed to extend from the bottom of the rock socket up to the head of the pipe or part of the way up the casing, as determined by analysis. Figure 1810.3.9.6 depicts a socketed drilled shaft with a full-length structural core.

- The principal structural feature of the socketed drilled shaft is the rock socket, which is filled with concrete and designed to take the full load of the element by end bearing and the frictional resistance offered by the rough walls of the socket.

18-68

2012 INTERNATIONAL BUILDING CODE® COMMENTARY
The relationship between the depth of the socket and the bearing capacity of the rock cannot be determined with any great accuracy because of the natural joints, bedding planes and fissures generally found in rock formations. Experience has shown, however, that the frictional resistance in hard rock is often sufficient to carry the load. In the case of soft rock, the shear strength of the socket walls should be determined by testing rock samples. The purpose of the steel core is to bond with the concrete fill to act as a composite section and provide additional load-bearing capacity.

### 1810.3.10 Micropiles

Micropiles shall be designed and detailed in accordance with Sections 1810.3.10.1 through 1810.3.10.4.

- This section provides minimum requirements for the design and installation of micropiles. This type of pile has been a popular alternative to more conventional pile types where headroom or equipment access is otherwise limited.

#### 1810.3.10.1 Construction

Micropiles shall develop their load-carrying capacity by means of a bond zone in soil, bedrock or a combination of soil and bedrock. Micropiles shall be grouted and have either a steel pipe or tube or steel reinforcement at every section along the length. It shall be permitted to transition from deformed reinforcing bars to steel pipe or tube reinforcement by extending the bars into the pipe or tube section by at least their development length in tension in accordance with ACI 318.

- Section 1802.1 defines "Micropiles" as bored or grouted in-place elements. This section provides a description of micropile construction. It requires incorporating steel pipe casing or steel reinforcement. Either of which is required to extend the full length of the pile. Note the further requirement in Section 1810.3.10.4 for a permanent steel casing in structures that are classified as Seismic Design Category C. Steel pipe casings are typically manufactured in segments with threaded ends.

This section clarifies the intent to permit the pipe or tube casing to terminate above the bond zone, with deformed bar reinforcement continuing below. It also specifies a splice condition for that transition.

---

**Figure 1810.3.9.5**

**BELLED DRILLED SHAFT**

For SI: 1 degree = 0.01745 rad.

**Figure 1810.3.9.6**

**SOCKETED DRILLED SHAFTS**

For SI: 1 inch = 25.4 mm.
1810.3.10.2 Materials. Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150.

The steel pipe or tube shall have a minimum wall thickness of \( \frac{1}{16} \) inch (4.8 mm). Splices shall comply with Section 1810.3.6. The steel pipe or tube shall have a minimum yield strength of 45,000 psi (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40,000 pounds (18 160 kg) of pipe or tube.

This section provides the minimum specifications for the component materials of the micropile, such as reinforcing steel and steel pipe casing. It also establishes a minimum wall thickness for the steel pipe that is virtually identical to the limits for other steel pipe piles.

1810.3.10.3 Reinforcement. For micropiles or portions thereof grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or tube or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Micropiles or portions thereof grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe or tube is used for reinforcement, the portion of the grout enclosed within the pipe is permitted to be included in the determination of the allowable stress in the grout.

Microplies with steel pipe casing in place at the time the grout is placed are designed with 40 percent of the compression load carried by the steel casing or steel reinforcing. Micropiles with grout that is placed without a casing must be designed so that 100 percent of the compression load is carried by the reinforcing steel.

1810.3.10.4 Seismic reinforcement. For structures assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the micropile down to the point of zero curvature. For structures assigned to Seismic Design Category D, E or F, the micropile shall be considered as an alternative system in accordance with Section 104.11. The alternative system design, supporting documentation and test data shall be submitted to the building official for review and approval.

In structures that are classified as Seismic Design Category A or B, there are no additional micropile requirements. In structures that are classified as Seismic Design Category C, permanent steel pipe casing must be provided for the length of the micropile noted. In structures that are classified as Seismic Design Category D, or higher, these micropile provisions are not applicable; approval of micropiles can only be as an alternative method of design and construction.

1810.3.11 Pile caps. Pile caps shall be of reinforced concrete, and shall include all elements to which vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of vertical deep foundation elements shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of the elements. The tops of elements shall be cut or chipped back to sound material before capping.

Pile caps include all elements to which the piles are connected and are to be of reinforced concrete and designed in accordance with the requirements of ACI 318. For footings (pile caps) on piles, computations for moments and shears may be based on the assumption that the load reaction from any pile is concentrated at the pile center (see ACI 318 for loads and reactions of footings on piles).

The soil immediately under the pile cap is not considered to provide any support for vertical loads. The heads of all piles are to be embedded no less than 3 inches (76 mm) into pile caps and the edges of the pile caps are to extend at least 4 inches (102 mm) beyond the closest sides of all piles. The degree of fixity between a pile head and the concrete cap depends on the method of connection required to satisfy design considerations.

1810.3.11.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F, concrete deep foundation elements shall be connected to the pile cap by embedding the element reinforcement or field-placed dowels anchored in the element into the pile cap for a distance equal to their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension in the case of uplift, without reduction for excess reinforcement in accordance with Section 12.2.5 of ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the element shall be permitted provided the design is such that any hinging occurs in the confined region.

The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipes, tubes or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the development length in tension of the reinforcement.

This section describes special detailing requirements between the deep foundation element and pile cap, including required development of the longitudinal pile reinforcement in the cap and associated transverse reinforcement. This reinforcement is required to extend into the pile cap to tie the elements together and to assist in load transfer at the top of the element to the pile cap. The connection must consist
of embedment of the element reinforcement in the pile cap for a distance equal to the development length, as specified in ACI 318. Field-placed dowels anchored in the plastic concrete elements are acceptable. The development length to be provided is that for compression or, where uplift is indicated by analysis, tension without reduction in length for excess area. Where seismic confinement reinforcement at the top of the pile is required, alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile are permitted, provided the design would force the hinge to occur in the confined region.

1810.3.11.2 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, deep foundation element resistance to uplift forces or rotational restraint shall be provided by anchorage into the pile cap, designed considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the element in tension. Anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:
   1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element;
   1.2. The nominal tensile strength of a steel element; and
   1.3. The frictional force developed between the element and the soil multiplied by 1.3.

Exception: The anchorage is permitted to be designed to resist the axial tension force resulting from the seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7; or shall be capable of developing the full axial, bending and shear nominal strength of the element.

   Where the vertical lateral force-resisting elements are columns, the pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be designed to resist forces and moments that result from the application of seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

The requirements of this section address the need for conservatism in the design of connections between deep foundation elements and pile caps. They are intended to allow energy dissipating mechanisms, such as rocking, to occur in the soil without failure of the element.

This section requires that the pile cap flexural strength exceed that of the supported column flexural strength if the column is a part of the lateral-force-resisting system.

Additional requirements are specified in this section for batter piles in order to limit earthquake damage to these systems. By their nature, batter pile systems have limited ductility and have performed poorly under strong ground motions. They are required to be designed using the overstrength load combinations, which consider the maximum force expected to be developed in these elements during a seismic event.

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E or F, grade beams shall comply with the provisions in Section 21.12.3 of ACI 318 for grade beams, except where they are designed to resist the seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

This section references Chapter 21 of ACI 318 for designing the grade beams. It also allows grade beams to be designed to have the strength required by the overstrength load combinations instead of designing the grade beams as beams in accordance with Chapter 21 of ACI 318. The overstrength load combinations estimate the maximum forces that can realistically develop in the grade beam in an earthquake situation.

1810.3.13 Seismic ties. For structures assigned to Seismic Design Category C, D, E or F, individual deep foundations shall be interconnected by ties. Unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger pile cap or column design gravity load times the seismic coefficient, $S_{pz}$ divided by 10, and 25 percent of the smaller pile or column design gravity load.

Exception: In Group R-3 and U occupancies of light-frame construction, deep foundation elements supporting foundation walls, isolated interior posts detailed so the element is not subject to lateral loads or exterior decks and patios are not subject to interconnection where the soils are of adequate stiffness, subject to the approval of the building official.

The purpose of this section is to preclude excessive movement of one group of deep foundation elements with respect to another. The section is similar to Section 1809.13. One of the prerequisites of adequate structural performance during an earthquake is that the foundation of the structure must act as a unit. This is typically accomplished by tying together the pile caps of deep foundation elements with ties capable of carrying, both in tension and compression, a force equal to the lesser of 10 percent of the larger pile cap or column load multiplied by $S_{pz}$ or 25 percent of the smaller pile cap or column load. This can be accom-
plished through the use of equivalent types of restraint, such as a concrete floor slab, where restraint can be substantiated. Reliance upon passive soil pressure is limited to competent rock, dense granular soil or hard cohesive soil.

If soils are shown to be of adequate stiffness, the building official may allow the use of the exception for the following elements in Group R-3 and U occupancies not exceeding two stories of light-frame construction:

1. Elements supporting foundation walls.
2. Elements supporting isolated interior posts detailed so the pier is not subject to lateral loads.
3. Elements supporting lightly loaded exterior decks and patios.

1810.4 Installation. Deep foundations shall be installed in accordance with Section 1810.4. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section shall satisfy the applicable conditions of installation.

- These provisions establish minimum requirements for the installation of deep foundation elements.

1810.4.1 Structural integrity. Deep foundation elements shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of adjacent structures or of foundation elements being installed or already in place and as to avoid compacting the surrounding soil to the extent that other foundation elements cannot be installed properly.

- The placement of deep foundation elements is often by driving, vibrating, jacking, jetting, direct weight or a combination of these methods. Because of the generally harsh nature of deep foundation installation operations, elements can experience some degree of damage during placement; however, damage can be prevented or minimized by selecting the proper type of deep foundation placement methods and techniques, as well as the right equipment to accomplish the work, all based on adequate knowledge of the soil conditions obtained from a foundation investigation (see Section 1803.5.5).

Piles must be placed in a manner that maintains their structural integrity and installed to such depths as determined by foundation investigation and engineering analysis to safely resist the design loads that are to be imposed upon them. Care must be exercised during deep foundation placement operations to provide for the safety of adjacent piles or other structures—leaving their strength and load capacity unimpaired. Any pile damaged during installation to the extent that its structural integrity is affected must be satisfactorily repaired or rejected.

1810.4.1.1 Compressive strength of precast concrete piles. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the specified compressive strength ($f'_c$), but not less than the strength sufficient to withstand handling and driving forces.

- This section requires that precast reinforced concrete piles must not be driven until the concrete has acquired at least three-quarters of its specified compressive strength. For example, if a pile is manufactured at the minimum compressive strength of 4,000 psi (27.6 MPa), it must not be driven until it has obtained a strength of at least 3,000 psi (20.68 MPa) ($f'_c$ x 4,000). In all cases, however, concrete strength must have developed to the point that it is sufficient to sustain the stresses imposed on the pile by handling and driving operations.

1810.4.1.2 Casing. Where cast-in-place deep foundation elements are formed through unstable soils and concrete is placed in an open-drilled hole, a casing shall be inserted in the hole prior to placing the concrete. Where the casing is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the casing at a sufficient height to offset any hydrostatic or lateral soil pressure. Driven casings shall be mandrel driven their full length in contact with the surrounding soil.

- This section requires a casing where a cast-in-place deep foundation element is formed through unstable soil.

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within six element diameters center to center. In granular soils or within one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed element rises or drops, the element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

- Casings driven in granular soils are to be spaced at least six times the diameter of the element. Casings driven in cohesive soils are not to be spaced less than one-half of the element length when the concrete (in adjacent elements) is less than 48 hours old. This latter requirement reflects the fact that deep foundation elements driven in cohesive materials can cause significant soil displacement that can induce high lateral pressures on adjacent elements. Unless the concrete in these adjacent elements has had sufficient time to set (48 hours or more) and acquire some of its ultimate strength, considerable damage can be caused by the earth pressures. Because of the voids between the particles in granular soils, the effects of soil displacement are not as serious as for cohesive soils, such as clay.

The 48-hour concrete time-set requirement as stated in this section should not be confused with the 12-hour requirement referred to in Section 1810.4.8 for drilled or augered elements. While both types are cast-in-place concrete elements, unlike the driven
uncased elements, drilled or augered elements are not of the displacement type, which essentially compacts the soil and may cause great lateral pressures and earth movement. The soil in the drilled method of installation is removed rather than displaced, and the subsurface influence on adjacent deep foundation elements is far less than on the driven element, particularly in cohesive soils.

Driven uncased elements are exposed to various types of potential damage. Besides the possibility of concrete contamination by the surrounding soil and surface water at the top of the element, its cross section may be subjected to squeezing or necking from lateral soil pressures and intrusions of displaced soils or other obstructions. Also, the concrete may be damaged by loss of support caused by removal of adjacent element casings from the soil surrounding the element. Driven uncased deep foundation elements are not recommended under conditions where significant ground heave could occur or where highly unstable soils exist.

Subsurface investigations and load testing may be required to establish capacity of deep foundation element, since there can be no correlation between the driving resistance of the element casing and its capacity. This is because the casing is eventually removed.

1810.4.1.4 Driving near cased concrete. Deep foundation elements shall not be driven within four and one-half average diameters of a cased element filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in casings within heave range of driving.

*One advantage to this type of pile is that after the driving and removal of the mandrel before the concrete is placed, the steel casing can be inspected internally along its full length; therefore, any damage that has resulted from the pile-driving operation can be readily discovered and corrected.*

Steel casings are not to be driven closer than four and one-half pile diameters of any adjacent piles filled with concrete until the concrete in the shells has cured for at least 24 hours and achieved an early strength. The basic reason for requiring four and one-half pile diameters between driving and concreting is a general reluctance to expose fresh concrete to vibrations as it sets. Several independent tests have shown, however, that there is no detrimental effect caused by vibrations on setting concrete. In some cases, there was a strength gain. The code requirement of four and one-half diameters is considered reasonable. It is often impractical to place concrete in element shells right next to deep foundation elements being driven. Casings can be driven earlier than the minimum 24-hour period when approved by the building official.

Driven light-gage steel shells that are left open can also be damaged by earth pressures when driving other piles in the close vicinity. Under such circumstances, the shells are sometimes protected by inserting dummy mandrels.

Shells that are in place, adjacent to and within heave range of a deep foundation element being driven are normally left open if the soil condition is such that it can cause heave. This condition is particularly critical in cohesive soils, such as clay. Heaved elements must be redriven before filling with concrete. If a concreted element has heaved; however, it can be safely redriven if proper techniques and a suitably designed cushion are used.

For a general description of steel-cased cast-in-place concrete piles, refer to the commentary to Section 1810.3.1.6.

1810.4.1.5 Defective timber piles. Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

* A small drop in blow count is not usually a cause for concern and is not uncommon in layered soils. It is only a substantial change in the rate of pile penetration that is not related to soil properties, which indicates a potential problem. If the penetration resistance suddenly decreases, serious damage to the pile could have occurred (including breaking), and the condition must be investigated. The pile can be withdrawn and inspected, and if required, abandoned and replaced.

Another significant problem associated with the use of timber piles is the possibility of structural damage caused by overdriving. Overdriving usually occurs upon reaching rock or hard soil strata and may result in pile bending, brooming at the tip, crushing or brooming at the head or splitting or breaking along the pile section (see Figure 1810.4.1.5). To avoid such damage, if the penetration resistance suddenly increases, pile driving should be stopped immediately.

![Figure 1810.4.1.5](image_url)

**Figure 1810.4.1.5**

**Typical Effects of Overdriving Timber Piles**
SOILS AND FOUNDATIONS

1810.4.2 Identification. Deep foundation materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

- All deep foundation element materials must be identified for conformity to construction specifications, providing information such as strength (species and grade for timber piles) and dimensions, and other pertinent information as may be required. Such identification must be provided for all piers and piles, whether they are taken from the manufacturer’s stock or made for a particular project. Identification is to be maintained from the place of manufacture to the shipment, on-site handling, storage and installation of the deep foundation elements. Manufacturers, upon request, usually furnish certificates of compliance with construction specifications. In the absence of adequate data, piers and piles are to be tested to prove conformity to the specified grade.

In addition to mill certificates (steel piles), identification is made through plant manufacturing or inspection reports (precast concrete deep foundation elements and timber piles) and delivery tickets (concrete). Timber piles are stamped (branded) with information, such as producer, species, treatment and length.

Identification is essential when high-yield strength steel is specified. Frequently, pile cutoff lengths are reused and pile material may come from a jobber, a contractor’s yard or a material supplier. In such cases, mill certificates are not available and the steel should be tested to see if it complies with the specifications.

1810.4.3 Location plan. A plan showing the location and designation of deep foundation elements by an identification system shall be filed with the building official prior to installation of such elements. Detailed records for elements shall bear an identification corresponding to that shown on the plan.

- This section requires that a deep foundation element location plan clearly showing the designation of all piers or piles in the foundation system be submitted to the building official prior to installation of piles. Preferably, such plans should be submitted before delivery of the piers or piles to the construction site and prior to staking the deep foundation element locations.

The deep foundation element location plan is not only an important tool for the installation of piers or piles, but also serves to communicate information between the owner, engineer, contractor, manufacturer, special inspector, building official and other interested persons. The special inspector (see commentary, Section 1810.4.12) must keep piling logs, records and reports based on this identification system. The use of the deep foundation element plan is particularly important at sites where the variations in soil profiles are so extensive that it becomes necessary to manufacture piers or piles of different lengths to reach proper bearing levels. The building official should receive revised copies of the deep foundation element location plan whenever field changes are made that add, delete or relocate piles.

1810.4.4 Preexcavation. The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for deep foundation elements subject to load tests and in such a manner that will not impair the carrying capacity of the elements already in place or damage adjacent structures. Element tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.

- The use of preexcavation methods to facilitate the installation of deep foundation elements is often necessary for a number of reasons, including:

1. To drive elements through upper layers of hard soil;
2. To penetrate through subsurface obstructions;
3. To eliminate or reduce the possibility of ground heave that could result in lifting adjacent elements already driven;
4. To reduce ground pressures that could result in the lateral movement of adjacent piles or structures;
5. To reduce the amount of driving necessary to seat the elements in the required bearing stratum;
6. To reduce the possibility of damaging vibrations;
7. To reduce the amount of noise associated with pile-driving operations; and
8. To accommodate the placement of elements that are longer than the leads of the driving equipment.

Methods commonly used for preexcavation are jetting and predrilling. The jetting method has been found to be more effective in granular soils, such as sand and fine gravel, than in cohesive materials, such as clay. However, jetting should not be done in granular soils containing very coarse gravel, cobbles and small boulders because such material cannot be removed by the jet stream and will result in a collection of stones at the bottom of the hole, making it very difficult, if not impossible, to drive a pile through the mass of stone material. The jetting operations must be controlled to avoid excessive losses of soil that could affect the stability of adjacent structures or the required bearing capacity of previously installed piles.

Jetting operations must be carefully controlled to avoid excessive loss of soil, which could affect the load-bearing capacity of deep foundation elements.
already installed or the stability of adjacent structures. Deep foundation elements should be driven below the depth of the jetted hole until the required resistance or penetration is obtained. Before this preexcavation method is used, consideration should be given to the possibility that jetting, unless strictly controlled, can adversely affect load transfer, particularly as it involves the placement of nontapered piles.

In comparison to the jetting method described, a more controllable form of preexcavation is by predrilling or coring. This method greatly reduces the possibility of detrimental effects on adjacent piles or structures and can be performed as a dry operation or a wet rotary process. Dry drilling can be done by the use of a continuous-flight auger or a short-flight auger attached to the end of a drill stem or Kelly bar. Wet drilling requires a hollow-stem, continuous-flight auger or a hollow drill stem employing the use of spade bits. When the wet rotary process of predrilling is used, bentonite slurry or plain water is circulated to keep the hole open. As in the case of jetting, deep foundation elements should be driven with tips below the predrilled hole. This is necessary to prevent any voids or very loose or soft soils from occurring below the element tip.

There are other methods used for preexcavation purposes, such as the dry tube method and spudding, but such procedures are seldom used. The methods to be employed for preexcavation are subject to the approval of the building official.

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2. The installation of production elements shall be controlled according to power consumption, rate of penetration or other approved means that ensure element capacities equal or exceed those of the test elements.

The use of vibratory drivers for the installation of piles is not applicable to all types of soil conditions. They are most effective in granular soils with the use of nondisplacement piles, such as steel H-piles and pipe piles driven open ended. Vibratory drivers are also used for extracting piles or temporary casings employed in the construction of cast-in-place concrete piles.

Vibratory drivers, either low or high frequency, cause the pile to penetrate the soil by longitudinal vibrations. While this type of pile driver can produce remarkable results in the installation of nondisplacement piles under favorable soil conditions, the greatest difficulty is the lack of a reliable method of estimating the load-bearing capacity. After the pile has been installed with a vibratory driver, pile capacity can best be determined by using an impact-type hammer to set the pile in its final position.

An acceptable means of controlling pile capacity is by determining the power consumption in relation to the rate of penetration. Nonetheless, the use of a vibratory driver is only permitted where the pile load capacity is established by load tests in accordance with the requirements of Section 1810.3.3.1.2.

1810.4.6 Heaved elements. Deep foundation elements that have heaved during the driving of adjacent elements shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the element shall be verified by load tests in accordance with Section 1810.3.3.1.2.

When deep foundation elements are driven into cohesive soils, particularly saturated plastic clay materials, they often displace a volume of soil equal to that of the elements themselves. Such soil displacement usually occurs as ground heaves and may cause adjacent elements already driven to lift up and become unseated, causing a loss of load capacity. When this happens, heaved piles must be redriven to firm bearing in order to regain their required capacity. If heaved elements are not redriven, their capacities must be verified by means of load tests.

It should be noted that not all types of elements can be redriven. For example, heaved uncased cast-in-place concrete elements, or sectional elements whose splices cannot take tension, should be abandoned and replaced.

In redriving heaved elements, the same or comparable driving equipment should be employed as used in the original installation; however, there are exceptions to this rule. For example, during the installation of concrete-filled pipe elements, only the empty pipes were first driven. In redriving this type of element, the pipes may now be filled with concrete resulting in much stiffer and heavier sections than were driven initially. In such cases, the driving technique must be adjusted to accommodate a considerably lesser driving resistance.

One method commonly used to prevent or reduce objectionable displacements caused by deep foundation installations in soils subject to heaving is to preexcavate element holes in accordance with the requirements of Section 1810.4.4.

1810.4.7 Enlarged base cast-in-place elements. Enlarged bases for cast-in-place deep foundation elements formed by compacting concrete or by driving a precast base shall be formed in or driven into granular soils. Such elements shall be constructed in the same manner as successful prototype test elements driven for the project. Shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the shaft shall be filled sufficiently to reestablish lateral support by the soil. Where heave occurs, the element shall be replaced unless it is demonstrated that the element is undamaged and capable of carrying twice its design load.

An enlarged base cast-in-place deep foundation element is often installed by first driving a steel casing into the ground. The casing can either be temporary or permanent. After the required depth has been reached, the enlarged base is formed in the granular
bearing soil by progressively adding and driving out small batches of zero-slump concrete using a drop weight.

Another method of installing an enlarged base element is by use of an enlarged precast concrete base at the end of a mandrel-driven steel casing. A major problem associated with this type of installation is that, in driving the permanent casing into the ground, the precast base creates a hole that is larger than the diameter of the shaft, leaving an open space around the element and thus losing the lateral support usually provided by the surrounding soil. In such cases, the annular space around the shaft must be filled to provide the necessary lateral support. The customary practice is to fill the annular space by pumping grout or washing in granular material. An illustration of these enlarged base elements is provided in Figure 1810.4.7.

The spacings of enlarged base piles are normally greater than most other types of conventional piles in order to avoid base interferences and the close overlapping of soil-bearing areas that would serve to reduce pile capacity.

There are many potential problems associated with drilled-hole piles. Most of these problems are related to soil conditions, including soil or rock debris accumulating at the base of the pile or occurring in the pile shaft; reductions in the shaft cross section caused by the necking of soil walls because of soft materials or earth pressures; discontinuities in the pile shaft; hollows on the surface of the shaft and other problems linked to drilling operations. Such problems are usually addressed by the use of proper installation techniques.

Whenever unstable soils are encountered in drilling operations, such as loose granular soils, organic soils, very soft silts or clays and water-bearing subsurface materials, a temporary steel liner is to be placed in the hole to prevent the collapse of the earth walls or sloughing off of the soil during concrete placement.

In placing concrete in temporarily lined holes, the top of the concrete should be kept well above the bottom edge of the steel liner as it is withdrawn in order to offset any hydrostatic or lateral soil pressures. Stiff (low slump) concrete should not be used, so as to avoid the potential problem of concrete arching in the liner tube and causing discontinuities or voids to occur in the pile shaft as the liner is withdrawn. For a discussion on concrete consistency (slump values), see the commentary to Section 1808.8.1.

While many of the problems associated with auger-placed concrete piles are similar to those described above for drilled-hole piles, other problems may also be introduced because of the particular installation technique. The pile installation procedure using a hollow stem continuous-flight auger, both for drilling and concrete placement purposes, does not permit the use of temporary steel liners. Damage to this type of pile, because of improper installation procedures, can result in the incomplete filling of the pile hole, concrete discontinuities along the pile shaft, reductions in the cross-sectional area of the shaft (necking) and soil inclusions. Also, the loss of side support of the drilled hole or vertical displacement caused by
ground pressures or soil movement may cause serious damage to the pile shaft.

In drilling the pile hole, the hollow-stem auger should be rotated and advanced continuously until the required tip elevation is reached. At that point in the drilling operation, rotation of the auger should be stopped to avoid removing excess soils that could result in damaging adjacent piles and to keep the auger flights full of soil as a means of retaining the hole walls. Concrete is then pumped through the hollow stem, filling the hole from the bottom up as the auger is withdrawn.

The volume of placed concrete must be monitored and it should equal or exceed the theoretical volume of the augered hole. An excess concrete volume of 10 percent is not uncommon, but even larger excesses can be expected where the soils are more porous. If the amount of the concrete pumped into the hole is considerably more than the anticipated volume, then the cause should be investigated. It could mean, for instance, that concrete (grout) is being pumped into soft soil strata, solution cavities, underground pipelines or other underground structures.

Concrete should be pumped under continuous pressure and the rate of withdrawal of the auger should be carefully controlled for a continuous and full-sized shaft. Discontinuities in the concrete shaft can develop if the auger is improperly withdrawn. A smooth, continuous auger withdrawal should help to prevent the surrounding soil from squeezing into the hole (necking) and possibly contaminating the concrete.

Concrete pumping pressures at the auger outlet should be greater than any hydrostatic or lateral pressures occurring in the hole. Excessively high pumping pressures, however, should be avoided while placing concrete surrounded by soft soils because it could cause upward or lateral movement of adjacent piles.

If, for any reason, the pile-concreting operation is interrupted, a pressure drop occurs while pumping the concrete, the auger is improperly handled and is raised too fast or for any other cause that could damage the pile or cause a reduction in the required size of the pile section, then the hollow-stem auger must be redrilled to the required tip elevation and the pile reformed from the bottom up.

Unless approved by the building official, no new pile holes are to be drilled or injected with concrete if they are located within a center-to-center distance of six pile diameters from other adjacent piles containing fresh concrete that have not been allowed to set for a period of 12 hours or more. This is to reduce the possibility of damage to adjacent piles. If the concrete surface of a completed pile is observed to drop below its cast elevation as a result of the drilling of adjacent piles, then the completed pile should be rejected and replaced. Such an occurrence could indicate some deformation along the pile shaft that could affect its structural integrity.

1810.4.9 Socketed drilled shafts. The rock socket and pipe or tube casing of socketed drilled shafts shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket.

A brief description of the installation sequence of caisson piles is given in the commentary to Section 1810.3.9.6. One of the problems in the construction of caisson piles is that some water will seep into the bottom of the pile opening through the rock joints or fissures in the socket. Sometimes water may seep into the opening at the end of the pipe casing because of incomplete seating of the pipe into the rock material. If the water in the hole cannot be controlled by ejection or other methods rendering the placement of concrete under reasonably dry conditions, the concrete may be placed by a tremie or other methods when approved by the building official.

1810.4.10 Micropiles. Micropile deep foundation elements shall be permitted to be formed in holes advanced by rotary or percussive drilling methods, with or without casing. The elements shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the element until grout of suitable quality returns at the top of the element. The following requirements apply to specific installation methods:

1. For micropiles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the element to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to verify that the flow of grout inside the casing is not obstructed.

2. For a micropile or portion thereof grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.

3. For micropiles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.

4. Subsequent micropiles shall not be drilled near elements that have been grouted until the grout has had sufficient time to harden.

5. Micropiles shall be grouted as soon as possible after drilling is completed.

6. For micropiles designed with a full-length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means employed to assure grout coverage outside the casing.

This section provides the basic micropile installation requirements. Micropile boreholes are typically advanced by either of the listed methods, rotary drilling or rotary percussive drilling. Installation requirements differ based on whether a steel casing is permanent.
or temporary or not provided (Item 6, 1 or 2, respectively).

1810.4.11 Helical piles. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as determined by a registered design professional. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile.

- Helical pile shafts are screwed into the ground by application of torsion (also see ICC Evaluation Service AC 358).

1810.4.12 Special inspection. Special inspections in accordance with Sections 1705.7 and 1705.8 shall be provided for driven and cast-in-place deep foundation elements, respectively. Special inspections in accordance with Section 1705.9 shall be provided for helical piles.

- This section requires special inspections (see definition, Section 1702) in accordance with Sections 1704.8 and 1704.9 to verify proper and safe installations. To be approved by the building official, an inspector should be experienced in deep foundation element installation work. An inspector must be able to read and understand foundation plans and specifications, maintain accurate records, have a thorough understanding of the scope of the pile work to be performed, give concise and timely reports to the owner or engineer and submit in writing to the building official whatever field information is required.

- The duties of the deep foundation element inspector include the inspection and approval of pile-driving equipment; the inspection and acceptance of piles furnished for the work; the verification of all pile location stakes and spacings; the observation and recording of required information relating to the installation of the deep foundation element foundation, as well as keeping a log for each pile showing either the number of hammer blows for each foot of penetration and the final penetration in blows per inch or the driving record for only the last few feet of penetration. The driving log should also include information on the duration and cause of any delays, such as splicing time, equipment breakdown, changing cushions, etc. The record should also show the final tip and butt (cutoff) elevations, as well as the depths of preexcavation holes (see Section 1810.4.4). Finally, there should also be a record of any pile damage and repair work; pile extractions and replacements; observations on pile heaving and depth of redriving and information on pile alignment.

Bibliography

The following resource materials are referenced in this chapter or are relevant to the subject matter addressed in this chapter.

ACI 229R-99, Controlled Low-strength Materials. Farmington Hills, MI: American Concrete Institute, 1999.


ACI 318-11, Building Code Requirements for Structural Concrete. Farmington Hills, MI: American Concrete Institute, 2011.


