CHAPTER 19
CONCRETE

Italics are used for text within Sections 1902 through 1908 of this code to indicate provisions that differ from ACI 318.

SECTION 1901
GENERAL

1901.1 Scope. The provisions of this chapter shall govern the materials, quality control, design and construction of concrete used in structures.

1901.2 Plain and reinforced concrete. Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318 as amended in Section 1908 of this code. Except for the provisions of Sections 1904 and 1911, the design and construction of slabs-on-grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil.

1901.3 Source and applicability. The contents of Sections 1902 through 1907 of this chapter are patterned after, and in general conformity with, the provisions for structural concrete in ACI 318. Where sections within Chapters 2 through 7 of ACI 318 are referenced in other chapters and appendices of ACI 318, the provisions of Sections 1902 through 1907 of this code shall apply.

1901.4 Construction documents. The construction documents for structural concrete construction shall include:

1. The specified compressive strength of concrete at the stated ages or stages of construction for which each concrete element is designed.
2. The specified strength or grade of reinforcement.
3. The size and location of structural elements and reinforcement.
4. Provision for dimensional changes resulting from creep, shrinkage and temperature.
5. The magnitude and location of prestressing forces.
6. Anchorage length of reinforcement and location and length of lap splices.
7. Type and location of mechanical and welded splices of reinforcement.
8. Details and location of contraction or isolation joints specified for plain concrete.
9. Minimum concrete compressive strength at time of posttensioning.
10. Stressing sequence for posttensioning tendons.
11. For structures assigned to Seismic Design Category D, E or F, a statement if slab on grade is designed as a structural diaphragm. See Section 21.8.3.4 of ACI 318.

1901.5 Special inspection. The special inspection of concrete elements of buildings and structures and concreting operations shall be as required by Chapter 17.

SECTION 1902
DEFINITIONS

1902.1 General. The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meanings shown herein.

ADMIXTURE. Material other than water, aggregate or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.

AGGREGATE. Granular material, such as sand, gravel, crushed stone and iron blast-furnace slag, used with a cementing medium to form a hydraulic cement concrete or mortar.

AGGREGATE, LIGHTWEIGHT. Aggregate with a dry, loose weight of 70 pounds per cubic foot (pcf) (1120 kg/m^3) or less.

CEMENTITIOUS MATERIALS. Materials as specified in Section 1903 that have cementing value when used in concrete either by themselves, such as portland cement, blended hydraulic cements, and expansive cement, or such materials in combination with fly ash, other raw or calcined natural pozzolans, silica fume, and/or ground granulated blast-furnace slag.

COLUMN. A member with a ratio of height-to-least-lateral dimension exceeding three, used primarily to support axial compressive load.

CONCRETE. A mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures.

CONCRETE, SPECIFIED COMPRESSIVE STRENGTH OF, (f'_c). The compressive strength of concrete used in design and evaluated in accordance with the provisions of Section 1905, expressed in pounds per square inch (psi) (MPa). Whenever the quantity f'_c is under a radical sign, the square root of the numerical value only is intended, and the result has units of pounds per square inch (psi) (MPa).

CONTRACTION JOINT. Formed, sawed or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

DEFORMED REINFORCEMENT. Deformed reinforcing bars, bar mats, deformed wire, welded plain wire fabric and welded deformed wire fabric conforming to ACI 318, Section 3.5.3.

EFFECTIVE DEPTH OF SECTION (d). The distance measured from extreme compression fiber to the centroid of tension reinforcement.

ISOLATION JOINT. A separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

PEDESTAL. An upright compression member with a ratio of unsupported height to average least lateral dimension of three or less.

PLAIN CONCRETE. Structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

PLAIN REINFORCEMENT. Reinforcement that does not conform to the definition of deformed reinforcement. (See ACI 318, Section 3.5.4.)

PRECAST CONCRETE. A structural concrete element cast elsewhere than its final position in the structure.

PRESTRESSED CONCRETE. Structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

REINFORCED CONCRETE. Structural concrete reinforced with no less than the minimum amounts of prestressing tendons or nonprestressed reinforcement specified in ACI 318, Chapters 1 through 21 and Appendices A through C.

REINFORCEMENT. Material that conforms to Section 1903.5, excluding prestressing tendons unless specifically included.

RESHORES. Shores placed snugly under a concrete slab or other structural member after the original forms and shores have been removed from a larger area, thus requiring the new slab or structural member to deflect and support its own weight and existing construction loads applied prior to the installation of the reshores.

SHORES. Vertical or inclined support members designed to carry the weight of the formwork, concrete and construction loads above.

SPIRAL REINFORCEMENT. Continuously wound reinforcement in the form of a cylindrical helix.

STIRRUP. Reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to those in compression members.)

STRUCTURAL CONCRETE. Concrete used for structural purposes, including plain and reinforced concrete.

TENDON. A steel element such as wire, cable, bar, rod, or strand, or a bundle of such elements, used to impart prestress to concrete.

SECTION 1903
SPECIFICATIONS FOR TESTS AND MATERIALS

1903.1 General. Materials used to produce concrete and testing thereof shall comply with the applicable standards listed in ACI 318 and this section. *Tests of concrete and the materials used in concrete shall be in accordance with ACI 318 Section 3.8. Where required, special inspections and tests shall be in accordance with Chapter 17.*

1903.2 Cement. Cement used to produce concrete shall comply with ACI 318, Section 3.2. *In addition to the cements permitted by ACI 318, cement complying with ASTM C 1157 is permitted.*

1903.3 Aggregates. Aggregates used in concrete shall comply with ACI 318, Section 3.3.

1903.4 Water. Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials or other substances that are deleterious to concrete or steel reinforcement and shall comply with ACI 318, Section 3.4.

1903.5 Steel reinforcement. Reinforcement and welding of reinforcement to be placed in concrete construction shall conform to the requirements of this section.

1903.5.1 Reinforcement type. Reinforcement shall be deformed reinforcement, except plain reinforcement is permitted for spirals or tendons, and reinforcement consisting of structural steel, steel pipe or steel tubing is permitted where specified in ACI 318. Reinforcement shall comply with ACI 318, Section 3.5.

1903.5.2 Welding. Welding of reinforcing bars shall conform to AWS D1.4. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. The ASTM reinforcing bar specifications, except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to the requirements in AWS D1.4.

1903.6 Admixtures. Admixtures to be used in concrete shall be subject to prior approval by the registered design professional and shall comply with ACI 318, Section 3.6.

1903.7 Storage of materials. The storage of materials for use in concrete shall comply with the provisions of Sections 1903.7.1 and 1903.7.2.

1903.7.1 Manner of storage. Cementitious materials and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter.

1903.7.2 Unacceptable material. Any material that has deteriorated or has been contaminated shall not be used for concrete.

SECTION 1904
DURABILITY REQUIREMENTS

1904.1 Water-cementitious materials ratio. The water-cementitious materials ratios specified in Tables 1904.2.2 and 1904.3 shall be calculated using the weight of cement meeting ASTM C 150, ASTM C 595, ASTM C 845 or *ASTM C 1157*, plus the weight of fly ash and other pozzolans meeting ASTM C 618, slag meeting ASTM C 989, and silica fume meeting ASTM C 1240, if any, except that where concrete is exposed to deicing chemicals, Section 1904.2.3 further limits the amount of fly ash, pozzolans, silica fume, slag or the combination of these materials.

1904.2 Freezing and thawing exposures. Concrete that will be exposed to freezing and thawing or deicing chemicals shall comply with Sections 1904.2.1 through 1904.2.3.

1904.2.1 Air entrainment. Normal weight and lightweight concrete exposed to freezing and thawing or deicing chemicals shall be air-entrained with air content indicated in Table 1904.2.1. Tolerance on air content as delivered shall be ±1.5 percent. For specified compressive strength (f'_c) greater than 5,000 psi (34.47 MPa), reduction of air content indicated in Table 1904.2.1 by 1.0 percent is permitted.

TABLE 1904.2.1
TOTAL AIR CONTENT FOR FROST-RESISTANT CONCRETE

NOMINAL MAXIMUM AGGREGATE SIZE[a] (inches)	AIR CONTENT (percent)	
	Severe exposure[b]	Moderate exposure[b]
$^3/_8$	$7^1/_2$	6
$^1/_2$	7	$5^1/_2$
$^3/_4$	6	5
1	6	$4^1/_2$
$1^1/_2$	$5^1/_2$	$4^1/_2$
2[c]	5	4
3[c]	$4^1/_2$	$3^1/_2$

For SI: 1 inch = 25.4 mm.

a. See ASTM C 33 for tolerance on oversize for various nominal maximum size designations.

b. The severe and moderate exposures referenced in this table are not based on the weathering regions shown in Figure 1904.2.2. For the purposes of this section, severe and moderate exposures shall be defined as follows:

1. Severe exposure occurs where concrete will be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, parking garages and water tanks.
2. Moderate exposure occurs where concrete will be only occasionally exposed to moisture prior to freezing, and where deicing salts are not used. Examples are certain exterior walls, beams, girders and slabs not in direct contact with soil.

c. These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than $1^1/_2$ inches is removed by hand-picking or sieving and air content is determined on the minus $1^1/_2$-inch fraction of the mix (tolerance on air content as delivered applies to this value). Air content of total mix is computed from value determined on the minus $1^1/_2$-inch fraction.

1904.2.2 Concrete properties. Concrete that will be subject to the exposures given in Table 1904.2.2(1) shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of Section 1904.2.3.

Exception: *For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories in height, normal-weight aggregate concrete that is subject to weathering (freezing and thawing), as determined from Figure 1904.2.2, or deicer chemicals shall comply with the requirements of Table 1904.2.2(2).*

1904.2.3 Deicing chemicals. For concrete exposed to deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume, or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials given in Table 1904.2.3.

1904.3 Sulfate exposures. Where concrete will be exposed to sulfate-containing solutions, it shall comply with the provisions of Sections 1904.3.1 and 1904.3.2.

1904.3.1 Concrete quality. Concrete to be exposed to sulfate-containing solutions or soils shall conform to the requirements of Table 1904.3 or shall be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength from Table 1904.3.

1904.3.2 Calcium chloride. Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions as defined in Table 1904.3.

1904.4 Corrosion protection of reinforcement. Reinforcement in concrete shall be protected from corrosion and exposure to chlorides as provided by Sections 1904.4.1 and 1904.4.2.

1904.4.1 General. For corrosion protection of reinforcement in concrete, the maximum water-soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures shall not exceed the limits of Table 1904.4.1. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to ASTM C 1218.

1904.4.2 Exposure to chlorides. Where concrete with reinforcement will be exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater or spray from these sources, the requirements of Table 1904.2.2(1) for water-cementitious materials ratio and concrete strength, and the minimum concrete cover requirements of Section 1907.7 shall be satisfied. See ACI 318, Section 18.14, for corrosion protection of unbonded prestressing tendons.

SECTION 1905
CONCRETE QUALITY, MIXING AND PLACING

1905.1 General. The required strength and durability of concrete shall be determined by compliance with the proportioning, testing, mixing and placing provisions of Sections 1905.1.1 through 1905.13.

1905.1.1 Strength. Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 1905.3, as well as satisfy the durability criteria of Section 1904. Concrete shall be produced to minimize frequency of strengths below f'_c as prescribed in Section 1905.6.3.3. *For concrete designed and constructed in accordance with this chapter, f'_c shall not be less than 2,500 psi (17.22 MPa).*

1905.1.2 Cylinder tests. Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in Section 1905.6.3.

1905.1.3 Basis of f'_c. Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be as indicated in construction documents.

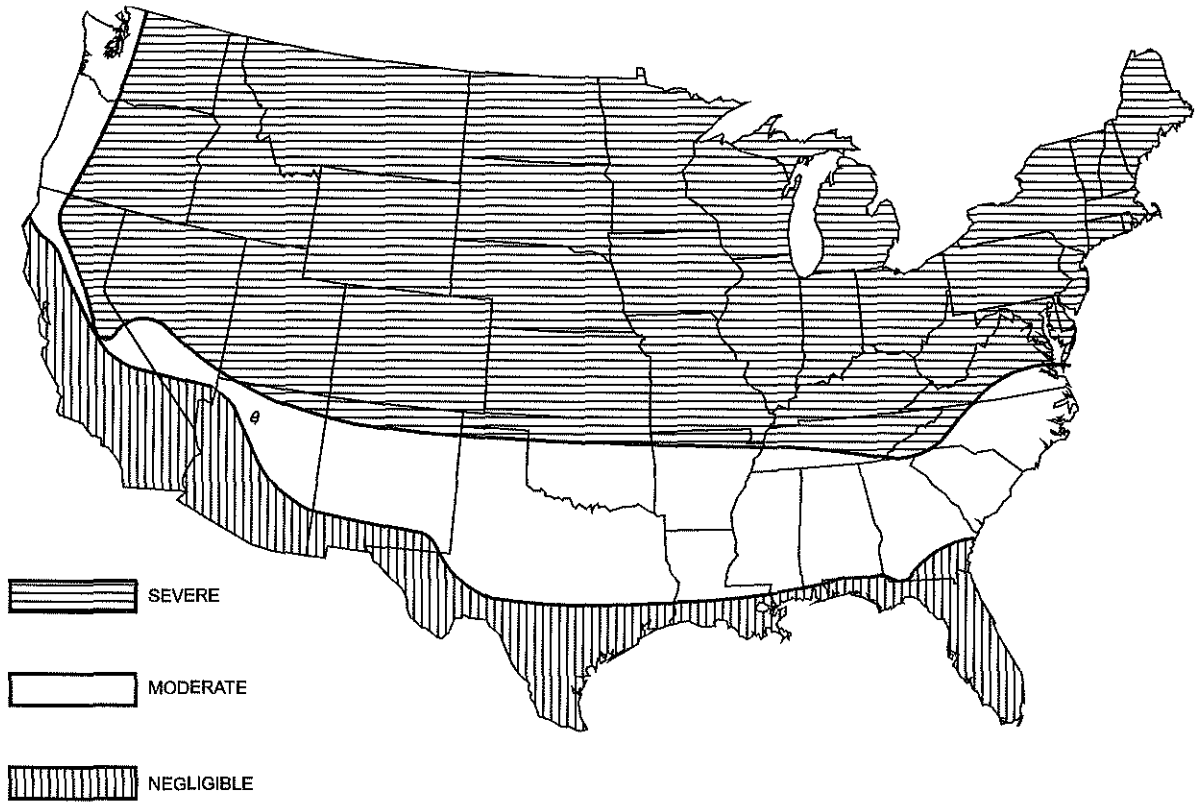


FIGURE 1904.2.2
WEATHERING PROBABILITY MAP FOR CONCRETE[a, b, c]

a. Lines defining areas are approximate only. Local areas can be more or less severe than indicated by the region classification.
b. A "severe" classification is where weather conditions encourage or require the use of deicing chemicals or where there is potential for a continuous presence of moisture during frequent cycles of freezing and thawing. A "moderate" classification is where weather conditions occasionally expose concrete in the presence of moisture to freezing and thawing, but where deicing chemicals are not generally used. A "negligible" classification is where weather conditions rarely expose concrete in the presence of moisture to freezing and thawing.
c. Alaska and Hawaii are classified as severe and negligible, respectively.

TABLE 1904.2.2(1)
REQUIREMENTS FOR SPECIAL EXPOSURE CONDITIONS

EXPOSURE CONDITION	MAXIMUM WATER-CEMENTITIOUS MATERIALS RATIO, BY WEIGHT, NORMAL WEIGHT AGGREGATE CONCRETE	MINIMUM f'_c, NORMAL WEIGHT AND LIGHTWEIGHT AGGREGATE CONCRETE (psi)
Concrete intended to have low permeability when exposed to water	0.50	4,000
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	4,500
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater or spray from these sources	0.40	5,000

For SI: 1 pound per square inch = 0.00689 MPa.

TABLE 1904.2.2(2)
MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f'_c)

TYPE OR LOCATION OF CONCRETE CONSTRUCTION	MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f'_c at 28 days, psi)		
	Negligible exposure	Moderate exposure	Severe exposure
Basement walls[c] and foundations not exposed to the weather	2,500	2,500	2,500[a]
Basement slabs and interior slabs on grade, except garage floor slabs	2,500	2,500	2,500[a]
Basement walls[c], foundation walls, exterior walls and other vertical concrete surfaces exposed to the weather	2,500	3,000[b]	3,000[b]
Driveways, curbs, walks, patios, porches, carport slabs, steps and other flatwork exposed to the weather, and garage floor slabs	2,500	3,000[b]	3,500[b]

For SI: 1 pound per square inch = 0.00689 MPa.

a. Concrete in these locations that can be subjected to freezing and thawing during construction shall be of air-entrained concrete in accordance with Table 1904.2.1.

b. Concrete shall be air-entrained in accordance with Table 1904.2.1.

c. Structural plain concrete basement walls are exempt from the requirements for special exposure conditions of Section 1904.2.2 (see Section 1909.1.1).

TABLE 1904.2.3
REQUIREMENTS FOR CONCRETE EXPOSED TO DEICING CHEMICALS

CEMENTITIOUS MATERIALS	MAXIMUM PERCENT OF TOTAL CEMENTITIOUS MATERIALS BY WEIGHT[a, b]
Fly ash or other pozzolans conforming to ASTM C 618	25
Slag conforming to ASTM C 989	50
Silica fume conforming to ASTM C 1240	10
Total of fly ash or other pozzolans, slag and silica fume	50[c]
Total of fly ash or other pozzolans and silica fume	35[c]

a. The total cementitious material also includes ASTM C 150, C 595, C 845 and C 1157 cement.

b. The maximum percentages shall include:

1. Fly ash or other pozzolans present in Type IP or I(PM) blended cement, ASTM C 595.
2. Slag used in the manufacture of an IS or I(SM) blended cement, ASTM C 595.
3. Silica fume, ASTM C 1240, present in a blended cement.

c. Fly ash or other pozzolans and silica fume shall constitute no more than 25 and 10 percent, respectively, of the total weight of the cementitious materials.

TABLE 1904.3
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

SULFATE EXPOSURE	WATER SOLUBLE SULFATE (SO_4) IN SOIL, PERCENT BY WEIGHT	SULFATE (SO_4) IN WATER (ppm)	CEMENT TYPE	MAXIMUM WATER-CEMENTITIOUS MATERIALS RATIO, BY WEIGHT, NORMAL WEIGHT AGGREGATE CONCRETE[a]	MINIMUM f'_c, NORMAL WEIGHT AND LIGHTWEIGHT AGGREGATE CONCRETE (psi)[a]
Negligible	0.00 - 0.10	0 - 150	—	—	—
Moderate[b]	0.10 - 0.20	150 - 1,500	II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4,000
Severe	0.20 - 2.00	1,500 - 10,000	V	0.45	4,500
Very severe	Over 2.00	Over 10,000	V plus pozzolan[c]	0.45	4,500

For SI: 1 pound per square inch = 0.00689 MPa.

a. A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (see Table 1904.2.2).

b. Seawater.

c. Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

TABLE 1904.4.1
MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION OF REINFORCEMENT

TYPE OF MEMBER	MAXIMUM WATER SOLUBLE CHLORIDE ION (CL) IN CONCRETE, PERCENT BY WEIGHT OF CEMENT
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

1905.1.4 Lightweight aggregate concrete. Where design criteria in ACI 318, Sections 9.5.2.3, 11.2 and 12.2.4, provide for use of a splitting tensile strength value of concrete (f_{ct}), laboratory tests shall be made in accordance with ASTM C 330 to establish the value of f_{ct} corresponding to the specified value of f'_c.

1905.1.5 Field acceptance. Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

1905.2 Selection of concrete proportions. Concrete proportions shall be determined in accordance with the provisions of Sections 1905.2.1 through 1905.2.3.

1905.2.1 General. Proportions of materials for concrete shall be established to provide:

1. Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under the conditions of placement to be employed, without segregation or excessive bleeding.
2. Resistance to special exposures as required by Section 1904.
3. Conformance with the strength test requirements of Section 1905.6.

1905.2.2 Different materials. Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

1905.2.3 Basis of proportions. Concrete proportions, including water-cementitious materials ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed in accordance with Section 1905.3, except as permitted in Section 1905.4, or required by Section 1904.

1905.3 Proportioning on the basis of field experience and/or trial mixtures. Concrete proportioning determined on the basis of field experience and/or trial mixtures shall be done in accordance with ACI 318, Section 5.3.

1905.4 Proportioning without field experience or trial mixtures. Concrete proportioning determined without field experience or trial mixtures shall be done in accordance with ACI 318, Section 5.4.

1905.5 Average strength reduction. As data become available during construction, it is permissible to reduce the amount by which the average compressive strength (f'_{cr}) is required to exceed the specified value of f'_c in accordance with ACI 318, Section 5.5.

1905.6 Evaluation and acceptance of concrete. The criteria for evaluation and acceptance of concrete shall be as specified in Sections 1905.6.2 through 1905.6.5.5.

1905.6.1 Qualified technicians. Concrete shall be tested in accordance with the requirements in Sections 1905.6.2 through 1905.6.5. Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens required for curing under field conditions, prepare specimens required for testing in the laboratory, and record the temperature of the fresh concrete when preparing specimens for strength tests. Qualified laboratory technicians shall perform all required laboratory tests.

1905.6.2 Frequency of testing. The frequency of conducting strength tests of concrete shall be as specified in Sections 1905.6.2.1 through 1905.6.2.4.

1905.6.2.1 Minimum frequency. Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 150 cubic yards (115 m^3) of concrete, nor less than once for each 5,000 square feet (465 m^2) of surface area for slabs or walls.

1905.6.2.2 Minimum number. On a given project, if the total volume of concrete is such that the frequency of testing required by Section 1905.6.2.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

1905.6.2.3 Small volume. When the total volume of a given class of concrete is less than 50 cubic yards (38 m^3), strength tests are not required when evidence of satisfactory strength is submitted to and approved by the building official.

1905.6.2.4 Strength test. A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at the test age designated for the determination of f'_c.

1905.6.3 Laboratory-cured specimens. Laboratory-cured specimens shall comply with the provisions of Sections 1905.6.3.1 through 1905.6.3.4.

1905.6.3.1 Sampling. Samples for strength tests shall be taken in accordance with ASTM C 172.

1905.6.3.2 Cylinders. Cylinders for strength tests shall be molded and laboratory cured in accordance with ASTM C 31 and tested in accordance with ASTM C 39.

1905.6.3.3 Acceptance of results. The strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

1. Every arithmetic average of any three consecutive strength tests equals or exceeds f'_c.
2. No individual strength test (average of two cylinders) falls below f'_c by more than 500 psi (3.45 MPa).

1905.6.3.4 Correction. If either of the requirements of Section 1905.6.3.3 is not met, steps shall be taken to increase the average of subsequent strength test results. The requirements of Section 1905.6.5 shall be observed if the requirement of Section 1905.6.3.3, Item 2 is not met.

1905.6.4 Field-cured specimens. Field-cured specimens shall comply with the provisions of Sections 1905.6.4.1 through 1905.6.4.4.

1905.6.4.1 When required. Where required by the building official, the results of strength tests of cylinders cured under field conditions shall be provided.

1905.6.4.2 Curing. Field-cured cylinders shall be cured under field conditions in accordance with ASTM C 31.

1905.6.4.3 Sampling. Field-cured test cylinders shall be molded at the same time and from the same samples as laboratory-cured test cylinders.

1905.6.4.4 Correction. Procedures for protecting and curing concrete shall be improved when the strength of field-cured cylinders at the test age designated for determination of f'_c is less than 85 percent of that of companion laboratory-cured cylinders. The 85 percent limitation shall not apply if the field-cured strength exceeds f'_c by more than 500 psi (3.45 MPa).

1905.6.5 Low-strength test results. The investigation of low-strength test results shall be in accordance with the provisions of Sections 1905.6.5.1 through 1905.6.5.5.

1905.6.5.1 Precaution. If any strength test (see Section 1905.6.2.4) of laboratory-cured cylinders falls below the specified value of f'_c by more than 500 psi (3.45 MPa) (see Section 1905.6.3.3, Item 2), or if tests of field-cured cylinders indicate deficiencies in protection and curing (see Section 1905.6.4.4), steps shall be taken to assure that the load-carrying capacity of the structure is not jeopardized.

1905.6.5.2 Core tests. If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with ASTM C 42 is permitted. In such cases, three cores shall be taken for each strength test more than 500 psi (3.45 MPa) below the specified value of f'_c.

1905.6.5.3 Condition of cores. If concrete in the structure will be dry under service conditions, cores shall be air dried at temperatures between 60°F (16°C) and 80°F (27°C) and relative humidity less than 60 percent for seven days before testing and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hours and be tested wet.

1905.6.5.4 Test results. Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f'_c and if no single core is less than 75 percent of f'_c. Additional testing of cores extracted from locations represented by erratic core strength results is permitted.

1905.6.5.5 Strength evaluation. If the criteria of Section 1905.6.5.4 are not met and if the structural adequacy remains in doubt, the building official is permitted to order a strength evaluation in accordance with ACI 318, Chapter 20, for the questionable portion of the structure, or take other appropriate action.

1905.7 Preparation of equipment and place of deposit. Preparation before concrete placement shall include the following:

1. Equipment for mixing and transporting concrete shall be clean.
2. Debris and ice shall be removed from spaces to be occupied by concrete.
3. Forms shall be properly coated.
4. Masonry filler units that will be in contact with concrete shall be well drenched.
5. Reinforcement shall be thoroughly clean of ice or other deleterious coatings.
6. Water shall be removed from the place of deposit before concrete is placed unless a tremie is to be used or unless otherwise permitted by the building official.
7. Laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

1905.8 Mixing. Mixing of concrete shall be performed in accordance with Sections 1905.8.1 through 1905.8.3.

1905.8.1 General. Concrete shall be mixed until there is a uniform distribution of materials and shall be discharged completely before the mixer is recharged.

1905.8.2 Ready-mixed concrete. Ready-mixed concrete shall be mixed and delivered in accordance with the requirements of ASTM C 94 or ASTM C 685.

1905.8.3 Job-mixed concrete. Job-mixed concrete shall comply with ACI 318, Section 5.8.3.

1905.9 Conveying. The method and equipment for conveying concrete to the place of deposit shall comply with Sections 1905.9.1 and 1905.9.2.

1905.9.1 Method of conveyance. Concrete shall be conveyed from the mixer to the place of final deposit by methods that will prevent separation or loss of materials.

1905.9.2 Conveying equipment. The conveying equipment shall be capable of providing a supply of concrete at the site of placement without separation of ingredients and without interruptions sufficient to permit the loss of plasticity between successive increments.

1905.10 Depositing. The depositing of concrete shall comply with the provisions of Sections 1905.10.1 through 1905.10.8.

1905.10.1 Segregation. Concrete shall be deposited as nearly as practicable to its final position to avoid segregation due to rehandling or flowing.

1905.10.2 Placement timing. Concreting operations shall be carried on at such a rate that the concrete is at all times plastic and flows readily into spaces between reinforcement.

1905.10.3 Unacceptable concrete. Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

1905.10.4 Retempering. Retempered concrete or concrete that has been remixed after initial set shall not be used unless approved by the registered design professional.

1905.10.5 Continuous operation. After concreting has started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed, except as permitted or prohibited by Section 1906.4.

1905.10.6 Placement in vertical lifts. The top surfaces of vertically formed lifts shall be generally level.

1905.10.7 Construction joints. When construction joints are required, they shall be made in accordance with Section 1906.4.

1905.10.8 Consolidation. Concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of the forms.

1905.11 Curing. The curing of concrete shall be in accordance with Sections 1905.11.1 through 1905.11.3.

1905.11.1 Regular. Concrete (other than high-early-strength) shall be maintained above 50°F (10°C) and in a moist condition for at least the first seven days after placement, except when cured in accordance with Section 1905.11.3.

1905.11.2 High-early-strength. High-early-strength concrete shall be maintained above 50°F (10°C) and in a moist condition for at least the first three days, except when cured in accordance with Section 1905.11.3.

1905.11.3 Accelerated curing. Accelerated curing of concrete shall comply with ACI 318, Section 5.11.3.

1905.12 Cold weather requirements. Concrete that is to be placed during freezing or near-freezing weather shall comply with the following:

1. Adequate equipment shall be provided for heating concrete materials and protecting concrete during freezing or near-freezing weather.
2. Concrete materials and reinforcement, forms, fillers and ground with which concrete is to come in contact shall be free from frost.
3. Frozen materials or materials containing ice shall not be used.

1905.13 Hot weather requirements. During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection and curing to prevent excessive concrete temperatures or water evaporation that could impair the required strength or serviceability of the member or structure.

SECTION 1906
FORMWORK, EMBEDDED PIPES AND CONSTRUCTION JOINTS

1906.1 Formwork. The design, fabrication and erection of forms shall comply with Sections 1906.1.1 through 1906.1.6.

1906.1.1 General. Forms shall result in a final structure that conforms to shapes, lines and dimensions of the members as required by the construction documents.

1906.1.2 Strength. Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

1906.1.3 Bracing. Forms shall be properly braced or tied together to maintain position and shape.

1906.1.4 Placement. Forms and their supports shall be designed so as not to damage previously placed structures.

1906.1.5 Design. Design of formwork shall comply with ACI 318, Section 6.1.5.

1906.1.6 Forms for prestressed concrete. Forms for prestressed concrete members shall be designed and constructed to permit movement of the member without damage during application of the prestressing force.

1906.2 Removal of forms, shores and reshores. The removal of forms and shores, and the installation of reshores shall comply with Sections 1906.2.1 through 1906.2.2.3.

1906.2.1 Removal of forms. Forms shall be removed in such a manner so as not to impair safety and serviceability of the structure. Concrete to be exposed by form removal shall have sufficient strength not to be not damaged by the removal operation.

1906.2.2 Removal of shores and reshores. The provisions of Sections 1906.2.2.1 through 1906.2.2.3 shall apply to slabs and beams, except where cast on the ground.

1906.2.2.1 Removal schedule. Before starting construction, the contractor shall develop a procedure and schedule for removal of shores and installation of reshores and for calculating the loads transferred to the structure during the process.

1. The structural analysis and concrete strength data used in planning and implementing form removal and shoring shall be furnished by the contractor to the building official when so requested.
2. No construction loads shall be supported on, nor any shoring removed from, any part of the structure under construction except when that portion of the structure in combination with the remaining forming and shoring system has sufficient strength to support safely its weight and the loads placed thereon.

3. Sufficient strength shall be demonstrated by structural analysis considering the proposed loads, the strength of the forming and shoring system, and concrete strength data. Concrete strength data shall be based on tests of field-cured cylinders or, when approved by the building official, on other procedures to evaluate concrete strength.

1906.2.2.2 Construction loads. No construction loads exceeding the combination of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

1906.2.2.3 Prestressed members. Form supports for prestressed concrete members shall not be removed until sufficient prestressing has been applied to enable prestressed members to carry their dead load and anticipated construction loads.

1906.3 Conduits and pipes embedded in concrete. Conduits, pipes and sleeves of any material not harmful to concrete and within the limitations of ACI 318, Section 6.3, are permitted to be embedded in concrete with approval of the registered design professional.

1906.4 Construction joints. Construction joints shall comply with the provisions of Sections 1906.4.1 through 1906.4.6.

1906.4.1 Surface cleaning. The surface of concrete construction joints shall be cleaned and laitance removed.

1906.4.2 Joint treatment. Immediately before new concrete is placed, construction joints shall be wetted and standing water removed.

1906.4.3 Location for force transfer. Construction joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for the transfer of shear and other forces through construction joints. See ACI 318, Section 11.7.9.

1906.4.4 Location in slabs, beams and girders. Construction joints in floors shall be located within the middle third of spans of slabs, beams and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

1906.4.5 Vertical support. Beams, girders or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic.

1906.4.6 Monolithic placement. Beams, girders, haunches, drop panels and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in the design drawings or specifications.

SECTION 1907
DETAILS OF REINFORCEMENT

1907.1 Hooks. Standard hooks on reinforcing bars used in concrete construction shall comply with ACI 318, Section 7.1.

1907.2 Minimum bend diameters. Minimum reinforcement bend diameters utilized in concrete construction shall comply with ACI 318, Section 7.2.

1907.3 Bending. The bending of reinforcement shall comply with Sections 1907.3.1 and 1907.3.2.

1907.3.1 Cold bending. Reinforcement shall be bent cold, unless otherwise permitted by the registered design professional.

1907.3.2 Embedded reinforcement. Reinforcement partially embedded in concrete shall not be field bent, except as shown on the construction documents or permitted by the registered design professional.

1907.4 Surface conditions of reinforcement. The surface conditions of reinforcement shall comply with the provisions of Sections 1907.4.1 through 1907.4.3.

1907.4.1 Coatings. At the time concrete is placed, reinforcement shall be free from mud, oil or other nonmetallic coatings that decrease bond. Epoxy coatings of steel reinforcement in accordance with ACI 318, Sections 3.5.3.7 and 3.5.3.8 are permitted.

1907.4.2 Rust or mill scale. Except for prestressing tendons, steel reinforcement with rust, mill scale or a combination of both, shall be considered satisfactory, provided the minimum dimensions, including height of deformations and weight of a hand-wire-brushed test specimen, comply with applicable ASTM specifications. See Section 1903.5.

1907.4.3 Prestressing tendons. Prestressing tendons shall be clean and free of oil, dirt, scale, pitting and excessive rust. A light coating of rust is permitted.

1907.5 Placing reinforcement. The placement of concrete reinforcement shall comply with the provisions of Sections 1907.5.1 through 1907.5.4.

1907.5.1 Support. Reinforcement, prestressing tendons, and ducts shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in Section 1907.5.2. *Where approved by the registered design professional,* embedded items (such as dowels or inserts) that either protrude from precast concrete members or remain exposed for inspection are permitted to be embedded while the concrete is in a plastic state, provided the following conditions are met:

1. Embedded items are not required to be hooked or tied to reinforcement within the concrete.
2. Embedded items are maintained in the correct position while the concrete remains plastic.
3. The concrete is properly consolidated around the embedded item.

1907.5.2 Tolerances. Unless otherwise specified by the registered design professional, reinforcement, prestressing tendons and prestressing ducts shall be placed within the tolerances specified in Sections 1907.5.2.1 and 1907.5.2.2.

1907.5.2.1 Depth and cover. Tolerance for depth *d*, and minimum concrete cover in flexural members, walls and compression members shall be as shown in Table 1907.5.2.1, except that tolerance for the clear distance to formed soffits shall be minus $^1/_4$ inch (6.4 mm) and tolerance for cover shall not exceed minus one-third the minimum concrete cover required in the design drawings or specifications.

1907.5.2.2 Bends and ends. Tolerance for longitudinal location of bends and ends of reinforcement shall be ± 2 inches (± 51 mm) except at discontinuous ends of members where the tolerance shall be ± $^1/_2$ inch (± 12.7 mm).

TABLE 1907.5.2.1
TOLERANCES

DEPTH (d) (inches)	TOLERANCE ON d (inch)	TOLERANCE ON MINIMUM CONCRETE COVER (inch)
$d > 8$	± $^3/_8$	– $^3/_8$
$d > 8$	± $^1/_2$	– $^1/_2$

For SI: 1 inch = 25.4 mm.

1907.5.3 Welded wire fabric. Welded wire fabric with wire size not greater than W5 or D5 used in slabs not exceeding 10 feet (3048 mm) in span is permitted to be curved from a point near the top of the slab over the support to a point near the bottom of the slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.

1907.5.4 Welding. Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the registered design professional.

1907.6 Spacing limits for reinforcement. The clear distance between reinforcing bars, bundled bars, prestressing tendons and ducts shall comply with ACI 318, Section 7.6.

1907.7 Concrete protection for reinforcement. The minimum concrete cover for reinforcement shall comply with Sections 1907.7.1 through 1907.7.7.

1907.7.1 Cast-in-place concrete (nonprestressed). Minimum concrete cover shall be provided for reinforcement in nonprestressed, cast-in-place concrete construction in accordance with Table 1907.7.1.

TABLE 1907.7.1
MINIMUM CONCRETE COVER

CONCRETE EXPOSURE	MINIMUM COVER (inches)
1. Concrete cast against and permanently exposed to earth	3
2. Concrete exposed to earth or weather	
No. 6 through No. 18 bar	2
No. 5 bar, W31 or D31 wire, and smaller	1$^1/_2$
3. Concrete not exposed to weather or in contact with ground	
Slabs, walls, joists:	
No. 14 and No. 18 bars	1$^1/_2$
No. 11 bar and smaller	$^3/_4$
Beams, columns:	
Primary reinforcement, ties, stirrups, spirals	1$^1/_2$
Shells, folded plate members:	
No. 6 bar and larger	$^3/_4$
No. 5 bar, W31 or D31 wire, and smaller	$^1/_2$

For SI: 1 inch = 25.4 mm.

1907.7.2 Precast concrete (manufactured under plant control conditions). The minimum concrete cover for reinforcement in precast concrete manufactured under plant control conditions shall comply with ACI 318, Section 7.7.2.

1907.7.3 Prestressed concrete. The minimum concrete cover for reinforcement in prestressed concrete shall comply with ACI 318, Section 7.7.3.

1907.7.4 Bundled bars. The minimum concrete cover for bundled bars shall comply with ACI 318, Section 7.7.4.

1907.7.5 Corrosive environments. In corrosive environments or other severe exposure conditions, the amount of concrete protection shall be suitably increased, and the denseness and nonporosity of the protecting concrete shall be considered, or other protection shall be provided.

1907.7.6 Future extensions. Exposed reinforcement, inserts and plates intended for bonding with future extensions shall be protected from corrosion.

1907.7.7 Fire protection. When this code requires a thickness of cover for fire protection greater than the minimum concrete cover specified in Section 1907.7, such greater thickness shall be used.

1907.8 Special reinforcement details for columns. Offset bent longitudinal bars in columns and load transfer in structural steel cores of composite compression members shall comply with the provisions of ACI 318, Section 7.8.

1907.9 Connections. Connections between concrete framing members shall comply with the provisions of ACI 318, Section 7.9.

1907.10 Lateral reinforcement for compression members. Lateral reinforcement for concrete compression members shall comply with the provisions of ACI 318, Section 7.10.

1907.11 Lateral reinforcement for flexural members. Lateral reinforcement for compression reinforcement in concrete flexural members shall comply with the provisions of ACI 318, Section 7.11.

1907.12 Shrinkage and temperature reinforcement. Reinforcement for shrinkage and temperature stresses in concrete members shall comply with the provisions of ACI 318, Section 7.12.

1907.13 Requirements for structural integrity. The detailing of reinforcement and connections between concrete members shall comply with the provisions of ACI 318, Section 7.13 to improve structural integrity.

SECTION 1908
MODIFICATIONS TO ACI 318

1908.1 General. The text of ACI 318 shall be modified as indicated in Sections 1908.1.1 through 1908.1.11.

1908.1.1 ACI 318, Section 8.1.2. Modify ACI 318, Section 8.1.2 to read as follows:

8.1.2 *Except for load combinations that include earthquake loads*, design of nonprestressed reinforced concrete members using Appendix A, Alternate Design Method, is permitted.

1908.1.2 ACI 318, Section 9.2.3. Modify Section 9.2.3 to read as follows:

9.2.3 Where resistance to specified earthquake loads or forces E are included in design, the load combinations of *Section 1605.2 of the* International Building Code *for strength design shall apply.*

1908.1.3 ACI 318, Section 18.9.3. Modify ACI 318 Section 18.9.3 to read as follows:

18.9.3 For two-way *slab systems*, minimum area and distribution of bonded reinforcement shall be as required in 18.9.3.1, 18.9.3.2, and 18.9.3.3.

1908.1.4 ACI 318, Section 21.0. Add the following notations to ACI 318, Section 21.0:

h = *Overall dimension of member in the direction of action considered.*

S_e *Connection = Moment, shear or axial force at connection cross section other than the nonlinear action location corresponding to probable strength at the nonlinear action location, taking gravity load effects into consideration per Section 21.2.8.3.*

S_n *Connection = Nominal strength of connection cross section in flexural, shear or axial action per Section 21.2.8.3.*

$$\Delta_m = C_d \Delta_s \qquad \textbf{(Equation 19-1)}$$

Δ_s = *Design level response displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.*

ψ = *Dynamic amplification factor from Sections 21.2.8.3 and 21.2.8.4.*

1908.1.5 ACI 318, Section 21.1. Modify existing definitions and add the following definitions to ACI 318, Section 21.1.

CONNECTION. *An element that joins two precast members or a precast member and a cast-in-place member.*

DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake, *as specified by Section 1617.4.6 or 1617.5.3 of the International Building Code.*

DESIGN LOAD COMBINATIONS. *Combinations of factored loads and forces specified in Section 1605.2 of the International Building Code.*

DRY CONNECTION. *Connection used between precast members that does not qualify as a wet connection.*

JOINT. *The geometric volume common to the intersecting members.*

NONLINEAR ACTION LOCATION. *Center of the region of yielding in flexure, shear or axial action.*

NONLINEAR ACTION REGION. *The member length over which nonlinear action takes place. It shall be taken as extending a distance of no less than h/2 on either side of the nonlinear action location.*

STRONG CONNECTION. *A connection that remains elastic while the designated nonlinear action regions undergo inelastic response under the design basis ground motion.*

WALL PIER. *A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*

WET CONNECTION. *A connection that uses any of the splicing methods permitted by Section 21.3.2.3 or 21.3.2.4 to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.*

1908.1.6 ACI 318, Section 21.2.1. Add new Sections 21.2.1.6 and 21.2.1.7:

21.2.1.6 Precast lateral-force-resisting systems shall satisfy either of the following criteria:

1. *It emulates the behavior of monolithic reinforced concrete construction and satisfies Section 21.2.2.5, or*
2. *It relies on the unique properties of a structural system composed of interconnected precast elements and it is demonstrated by experimental evidence and analysis to safely sustain the seismic loading requirements of a comparable monolithic reinforced concrete structure satisfying Chapter 21. Substantiating experimental evidence of acceptable performance of those elements required to sustain inelastic deformations shall be based on cyclic inelastic testing of specimens representing those elements.*

21.2.1.7 In structures having precast gravity load-carrying systems, the lateral-force-resisting system shall be one of the systems listed in Table 1617.6 of the International Building Code® and shall be well distributed using one of the following methods:

1. *The lateral-force-resisting system shall be spaced such that the span of the diaphragm or diaphragm segment between lateral-force-resisting systems shall be no more than three times the width of the diaphragm or diaphragm segment. Where the lateral-force-resisting system consists of moment-resisting frames, at least $(N_b/4) + 1$ of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the lateral-force-resisting system where N_b is the total number of bays along that line at that story. This requirement applies to only the lower two-thirds of the stories of buildings three stories or taller.*
2. *Beam-to-column connections that are not part of the lateral-force-resisting system shall be designed in accordance with the following:*

   Connection Design Force. *The connection shall be designed to develop strength M. M is the moment developed at the connection when the frame is displaced by Δ_s assuming fixity at the connection and a beam flexural stiffness of* *no less than one-half of the gross section stiffness. M shall be sustained through a deformation of Δ_m.*

Connection Characteristics. *The connection is permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member shall resist moment with the same positive or negative sign. The connection shall be permitted to have zero flexural stiffness up to a frame displacement of Δ_s.*

In addition, complete calculations for the deformation compatibility of the gravity load-carrying system shall be made in accordance with Section 1617.6.4.3 of the International Building Code® using cracked section stiffness in the lateral-force-resisting system and the diaphragm.

Where gravity columns are not provided with lateral support on all sides, a positive connection shall be provided along each unsupported direction parallel to a principal plan axis of the structure. The connection shall be designed for a horizontal force equal to 4 percent of the axial load strength, P_o, of the column.

The bearing length shall be calculated to include end rotation, sliding and other movements of precast ends at supports due to earthquake motions in addition to other movements and shall be at least 2 inches (51 mm) more than that required.

1908.1.7 ACI 318, Section 21.2.2. Add new Sections 21.2.2.5, 21.2.2.6 and 21.2.2.7 to ACI 318, Section 21.2.2 to read as follows:

21.2.2.5 Precast structural systems using frames and emulating the behavior of monolithic reinforced concrete construction shall satisfy either Section 21.2.2.6 or 21.2.2.7.

21.2.2.6 Precast structural systems utilizing wet connections shall comply with the applicable requirements of monolithic concrete construction for resisting seismic forces.

21.2.2.7 Precast structural systems not meeting the requirements of Section 21.2.2.6 shall utilize strong connections resulting in nonlinear response away from connections. Design shall satisfy the requirements of Section 21.2.8 in addition to the applicable requirements of monolithic concrete construction for resisting seismic forces, except that provisions of Section 21.3.1.2 shall apply to the segments between nonlinear action locations.

1908.1.8 ACI 318, Section 21.2.5. Modify ACI 318, Section 21.2.5 by renumbering as Section 21.2.5.1 and adding new Sections 21.2.5.2 and 21.2.5.3 to read as follows:

21.2.5 Reinforcement in members resisting earthquake-induced forces.

21.2.5.1 Except as permitted in Sections 21.2.5.2 through 21.2.5.3, reinforcement resisting earthquake-induced flexural and axial forces in frame members and in structural wall boundary elements shall comply with ASTM A 706. ASTM 615 Grades 40 and 60 reinforcement shall be permitted in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (retests shall not exceed this value by more than an additional 3,000 psi), and (b) the ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25.

21.2.5.2 Prestressing tendons shall be permitted in flexural members of frames, provided the average prestress, f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension shall be the lesser of 700 psi (4.83 MPa) or $f'_c/6$ at locations of nonlinear action where prestressing tendons are used in members of frames.

21.2.5.3 For members in which prestressing tendons are used together with mild reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more than one quarter of the strength for both positive moments and negative moments at the joint face and shall extend through exterior joints and be anchored at the exterior face of the joint or beyond. Anchorages for tendons must be demonstrated to perform satisfactorily for seismic loadings. Anchorage assemblies shall withstand, without failure, a minimum of 50 cycles of loading ranging between 40 and 85 percent of the minimum specified tensile strength of the tendon.

1908.1.9 ACI 318, Section 21.2. Modify ACI 318, Section 21.2 by adding a new Section 21.2.8 to read as follows:

21.2.8 Emulation of monolithic construction using strong connections. Members resisting earthquake-induced forces in precast frames using strong connections shall satisfy the following:

21.2.8.1 Location. Nonlinear action location shall be selected so that there is a strong column/weak beam deformation mechanism under seismic effects. The nonlinear action location shall be no closer to the near face of the strong connection than h/2. For column-to-footing connections where nonlinear action may occur at the column base to complete the mechanism, the nonlinear action location shall be no closer to the near face of the connection than h/2.

21.2.8.2 Anchorage and splices. Reinforcement in the nonlinear action region shall be fully developed outside both the strong connection region and the nonlinear action region. Noncontinuous anchorage reinforcement of the strong connection shall be fully developed between the connection and the beginning of the nonlinear action region. Lap splices are prohibited within connections adjacent to a joint.

21.2.8.3 Design forces. Design strength of strong connections shall be based on:

ϕS_n Connection > ψS_e Connection
Dynamic amplification factor, ψ, shall be taken as 1.0.

21.2.8.4 Column-to-column connection. The strength of column-to-column connections shall comply with Section 21.2.8.3 with ψ taken as 1.4. Where column-to-column connections occur, the columns shall be provided with transverse reinforcement as specified in Sections 21.4.4.1 through 21.4.4.3 over their full height if the factored axial compressive force in these members, including seismic effects, exceeds $A_g f'_c/10$.

> ***Exception:*** *Where the column-to-column connection is located within the middle third of the column clear height, the following shall apply: (a) the design moment strength, ϕM_n, of the connection shall not be less than 0.4 times the maximum M_{pr} for the column within the story height, and (b) the design shear strength, ϕV_n, of the connection shall not be less than that determined by Section 21.4.5.1.*

21.2.8.5 Column-face connection. Any strong connection located outside the middle half of a beam span shall be a wet connection unless a dry connection can be substantiated by approved cyclic test results. Any mechanical connector located within such a column-face strong connection shall be a Type 2 mechanical splice as defined in Section 21.2.6.

1908.1.10 ACI 318, Section 21.6. Modify ACI 318, Section 21.6 by adding a new Section 21.6.10 to read as follows:

21.6.10 Wall piers and Wall segments.

21.6.10.1 Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in Section 21.6.10.2.

> ***Exceptions:***
>
> 1. *Wall piers that satisfy Section 21.9.*
> 2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers, and such segments have a total stiffness of at least six times the sum of the stiffness of all the wall piers.*

21.6.10.2 Transverse reinforcement shall be designed to resist the shear forces determined from Sections 21.3.4.2 and 21.4.5.1. Where the axial compressive force, including earthquake effects, is less than $A_g f'_c/20$, transverse reinforcement in wall piers is permitted to have standard hooks at each end in lieu of hoops. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least the development length of the largest longitudinal reinforcement in the wall pier.

21.6.10.3 Wall segments with a horizontal length-to-thickness ratio less than $2^1/_2$, shall be designed as columns.

1908.1.11 ACI 318, Section 21.9.2.2. Modify ACI 318 Section 21.9.2.2 to read as follows:

21.9.2.2 Members with factored gravity axial forces exceeding ($A_g f'_c/10$) shall satisfy Sections *21.4.3.1*, 21.4.4.1(c), 21.4.4.3 and 21.4.5. The maximum longitudinal spacing of ties shall be s_o for the full column height. *The spacing s_o shall not be more than six diameters of the smallest longitudinal bar enclosed or 6 inches (152 mm), whichever is smaller.*

SECTION 1909
STRUCTURAL PLAIN CONCRETE

1909.1 Scope. The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of Section 1909 and ACI 318 Chapter 22.

1909.1.1 Special structures. For special structures, such as arches, underground utility structures, gravity walls and shielding walls, the provisions of this section shall govern where applicable.

1909.2 Limitations. The use of structural plain concrete shall be limited to:

1. Members that are continuously supported by soil, such as walls and footings, or by other structural members capable of providing continuous vertical support.
2. Members for which arch action provides compression under all conditions of loading.
3. Walls and pedestals.

The use of structural plain concrete columns and structural plain concrete footings on piles is not permitted. See Section 1910 for additional limitations on the use of structural plain concrete.

1909.3 Joints. Contraction or isolation joints shall be provided to divide structural plain concrete members into flexurally discontinuous elements in accordance with ACI 318, Section 22.3.

1909.4 Design. Structural plain concrete walls, footings and pedestals shall be designed for adequate strength in accordance with ACI 318, Sections 22.4 through 22.8.

> **Exception:** For Group R-3 as applicable in Section 101.2 occupancies and buildings of other occupancies less than two stories in height of light-frame construction, the required edge thickness of ACI 318 is permitted to be reduced to 6 inches (152 mm), provided that the footing does not extend more than 4 inches (102 mm) on either side of the supported wall.

1909.5 Precast members. The design, fabrication, transportation and erection of precast, structural plain concrete elements shall be in accordance with ACI 318, Section 22.9.

1909.6 Walls. In addition to the requirements of this section, structural plain concrete walls shall comply with the applicable requirements of ACI 318, Chapter 22.

1909.6.1 Basement walls. The thickness of exterior basement walls and foundation walls shall be not less than $7^1/_2$ inches (191 mm). Structural plain concrete exterior basement walls shall be exempt from the requirements for special exposure conditions of Section 1904.2.2.

1909.6.2 Other walls. Except as provided for in Section 1909.6.1, the thickness of bearing walls shall be not less than $^1/_{24}$ the unsupported height or length, whichever is shorter, but not less than $5^1/_2$ inches (140 mm).

1909.6.3 Openings in walls. Not less than two No. 5 bars shall be provided around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of openings.

SECTION 1910
SEISMIC DESIGN PROVISIONS

1910.1 General. The design and construction of concrete components that resist seismic forces shall conform to the requirements of this section and to ACI 318 except as modified by Section 1908.

1910.2 Classification of shear walls. Structural concrete shear walls that resist seismic forces shall be classified in accordance with Sections 1910.2.1 through 1910.2.4.

1910.2.1 Ordinary plain concrete shear walls. Ordinary plain concrete shear walls are walls conforming to the requirements of Chapter 22 of ACI 318.

1910.2.2 Detailed plain concrete shear walls. Detailed plain concrete shear walls are walls conforming to the requirements for ordinary plain concrete shear walls and shall have reinforcement as follows: Vertical reinforcement of at least 0.20 square inch (129 mm^2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for one of the two No. 5 bars required by Section 22.6.6.5 of ACI 318. Horizontal reinforcement at least 0.20 square inch (129 mm^2) in cross sectional area shall be provided:

1. Continuously at structurally connected roof and floor levels and at the top of walls;
2. At the bottom of load bearing walls or in the top of foundations where doweled to the wall; and
3. At a maximum spacing of 120 inches (3048 mm).

Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in item 3 above, shall be continuous in the wall.

1910.2.3 Ordinary reinforced concrete shear walls. Ordinary reinforced concrete shear walls are walls conforming to the requirements of ACI 318 for ordinary reinforced concrete structural walls.

1910.2.4 Special reinforced concrete shear walls. Special reinforced concrete shear walls are walls conforming to the requirements of ACI 318 for special reinforced concrete structural walls.

1910.3 Seismic Design Category B. Structures assigned to Seismic Design Category B, as determined in Section 1616, shall conform to the requirements for Seismic Design Category A and to the additional requirements for Seismic Design Category B of this section.

1910.3.1 Ordinary moment frames. In flexural members of ordinary moment frames forming part of the seismic-force-resisting system, at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams, through or developed within exterior columns or boundary elements.

Columns of ordinary moment frames having a clear height to maximum plan dimension ratio of five or less shall be designed for shear in accordance with Section 21.10.3 of ACI 318.

1910.4 Seismic Design Category C. Structures assigned to Seismic Design Category C, as determined in Section 1616 shall conform to the requirements for Seismic Design Category B and to the additional requirements for Seismic Design Category C of this section.

1910.4.1 Seismic-force-resisting systems. Moment frames used to resist seismic forces shall be intermediate moment frames or special moment frames. Shear walls used to resist seismic forces shall be ordinary reinforced concrete shear walls or special reinforced concrete shear walls.

1910.4.2 Discontinuous members. Columns supporting reactions from discontinuous stiff members, such as walls, shall be designed for the special load combinations in Section 1605.4 and shall be provided with transverse reinforcement at the spacing s_o as defined in Section 21.10.5.1 of ACI 318 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as required in Section 21.4.4.5 of ACI 318.

1910.4.3 Anchor bolts in the top of columns. Anchor bolts which are set in the top of a column shall be provided with ties which enclose at least four longitudinal column bars. There shall be at least two No. 4 (#13), or three No. 3 (#10) ties within 5 inches (127 mm) of the top of the column. The ties shall have hooks on each free end which comply with Section 7.1.3 (c) of ACI 318.

1910.4.4 Plain concrete. Structural plain concrete members in structures assigned to Seismic Design Category C shall conform to ACI 318 and with Sections 1910.4.4.1 through 1910.4.4.3.

1910.4.4.1 Walls. Structural plain concrete walls are not permitted in structures assigned to Seismic Design Category C.

Exception: Structural plain concrete basement, foundation or other walls below the base are permitted in detached one- and two-family dwellings constructed with stud-bearing walls. Such walls shall have reinforcement in accordance with Section 22.6.6.5 of ACI 318.

1910.4.4.2 Footings. Isolated footings of plain concrete supporting pedestals or columns are permitted provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

Exception: In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member is permitted to exceed the footing thickness.

Plain concrete footings supporting walls shall be provided with not less than two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings which exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. For foundation systems consisting of a plain concrete footing and a plain concrete stemwall, a minimum of one bar shall be provided at the top of the stemwall and at the bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

1. In detached one- and two-family dwellings three stories or less in height and constructed with stud-bearing walls, plain concrete footings supporting walls are permitted without longitudinal reinforcement.
2. Where a slab-on-ground is cast monolithically with the footing, one No. 5 bar is permitted to be located at either the top or bottom of the footing.

1910.4.4.3 Pedestals. Plain concrete pedestals shall not be used to resist lateral seismic forces.

1910.5 Seismic Design Category D, E or F. Structures assigned to Seismic Design Category D, E or F, as determined in Section 1616, shall conform to the requirements for Seismic Design Category C and to the additional requirements of this section.

1910.5.1 Seismic-force-resisting systems. Moment frames used to resist seismic forces shall be special moment frames. Shear walls used to resist seismic forces shall be special reinforced concrete shear walls.

1910.5.2 Frame members not proportioned to resist forces induced by earthquake motions. Frame components assumed not to contribute to lateral force resistance shall conform to ACI 318, Section 21.9, as modified by Section 1908.1.11 of this chapter.

SECTION 1911 MINIMUM SLAB PROVISIONS

1911.1 General. The thickness of concrete floor slabs supported directly on the ground shall not be less than $3^1/_2$ inches (89 mm). A 6-mil (0.006 inch; 152 μm) polyethylene vapor retarder with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other approved equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exception: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3 as applicable in Section 101.2, such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3 as applicable in Section 101.2.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork which will not be enclosed at a later date.
5. Where approved based on local site conditions.

SECTION 1912 ANCHORAGE TO CONCRETE— ALLOWABLE STRESS DESIGN

1912.1 Scope. The provisions of this section shall govern the allowable stress design of headed bolts and headed stud anchors cast in normal weight concrete for purposes of transmitting structural loads from one connected element to the other. These provisions do not apply to anchors installed in hardened concrete or where load combinations include earthquake loads or effects. The bearing area of headed anchors shall be not less than one and one-half times the shank area. Where strength design is used, or where load combinations include earthquake loads or effects, the design strength of anchors shall be determined in accordance with Section 1913. Bolts shall conform to ASTM A 307 or an approved equivalent.

1912.2 Allowable service load. The allowable service load for headed anchors in shear or tension shall be as indicated in Table 1912.2. Where anchors are subject to combined shear and tension, the following relationship shall be satisfied:

$$(P_s / P_t)^{5/3} + (V_s / V_t)^{5/3} \leq 1 \qquad \textbf{(Equation 19-2)}$$

where:

P_s = Applied tension service load, pounds (newtons).

P_t = Allowable tension service load from Table 1912.2, pounds (newtons).

V_s = Applied shear service load, pounds (newtons).

V_t = Allowable shear service load from Table 1912.2, pounds (newtons).

1912.3 Required edge distance and spacing. The allowable service loads in tension and shear specified in Table 1912.2 are for the edge distance and spacing specified. The edge distance and spacing are permitted to be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load shall be determined by linear interpolation.

1912.4 Increase in allowable load. Increase of the values in Table 1912.2 by one-third is permitted where the provisions of Section 1605.3.2 permit an increase in allowable stress for wind loading.

1912.5 Increase for special inspection. Where special inspection is provided for the installation of anchors, a 100-percent increase in the allowable tension values of Table 1912.2 is permitted. No increase in shear value is permitted.

TABLE 1912.2
ALLOWABLE SERVICE LOAD ON EMBEDDED BOLTS (pounds)

BOLT DIAMETER (inches)	MINIMUM EMBEDMENT (inches)	EDGE DISTANCE (inches)	SPACING (inches)	MINIMUM CONCRETE STRENGTH (psi)					
				f'_c = 2,500		f'_c = 3,000		f'_c = 4,000	
				Tension	Shear	Tension	Shear	Tension	Shear
$^1/_4$	$2^1/_2$	$1^1/_2$	3	200	500	200	500	200	500
$^3/_8$	3	$2^1/_4$	$4^1/_2$	500	1,100	500	1,100	500	1,100
$^1/_2$	4 4	3 5	6 5	950 1,450	1,250 1,600	950 1,500	1,250 1,650	950 1,550	1,250 1,750
$^5/_8$	$4^1/_2$ $4^1/_2$	$3^3/_4$ $6^1/_4$	$7^1/_2$ $7^1/_2$	1,500 2,125	2,750 2,950	1,500 2,200	2,750 3,000	1,500 2,400	2,750 3,050
$^3/_4$	5 5	$4^1/_2$ $7^1/_2$	9 9	2,250 2,825	3,250 4,275	2,250 2,950	3,560 4,300	2,250 3,200	3,560 4,400
$^7/_8$	6	$5^1/_4$	$10^1/_2$	2,550	3,700	2,550	4,050	2,550	4,050
1	7	6	12	3,050	4,125	3,250	4,500	3,650	5,300
$1^1/_8$	8	$6^3/_4$	$13^1/_2$	3,400	4,750	3,400	4,750	3,400	4,750
$1^1/_4$	9	$7^1/_2$	15	4,000	5,800	4,000	5,800	4,000	5,800

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 0.00689 MPa, 1 pound = 4.45 N.

SECTION 1913 ANCHORAGE TO CONCRETE—STRENGTH DESIGN

1913.1 Scope. The provisions of this section shall govern the strength design of anchors cast in concrete for purposes of transmitting structural loads from one connected element to the other. These provisions apply to headed bolts, headed studs, and hooked (J- or L-) bolts. These provisions do not apply to anchors installed in hardened concrete, or load applications that are predominantly high cycle fatigue or impact. The heads of headed studs and headed bolts shall have a geometry such that the pullout strength of the anchor in uncracked concrete, as demonstrated by approved tests, equals or exceeds 1.4 N_p [where N_p is given by Equation 19-17]. Hooked bolts shall have a geometry such that the pullout strength of the anchor without the benefit of friction in uncracked concrete, as demonstrated by approved tests, equals or exceeds 1.4 N_p [where N_p is given by Equation 19-18]. Reinforcement used as part of the embedment shall be designed in accordance with applicable parts of ACI 318.

1913.2 Notations and definitions. The notations and definitions used in this section shall be as set forth in Sections 1913.2.1 and 1913.2.2, respectively.

1913.2.1 Notations.

A_b = Bearing area of the head of stud or anchor bolt, inches squared.

A_{No} = Projected concrete failure area of one anchor, for calculation of strength in tension, when not limited by edge distance or spacing, as defined in Section 1913.5.2.1, inches squared.

A_N = Projected concrete failure area of an anchor or group of anchors, for calculation of strength in tension, as defined in Section 1913.5.2.1, inches squared. A_N shall not be taken greater than nA_{No}.

A_{se} = Effective cross-sectional area of anchor, inches squared.

A_{Vo} = Projected concrete failure area of one anchor, for calculation of strength in shear, when not limited by corner influences, spacing, or member thickness, as defined in Section 1913.6.2.1, inches squared.

A_V = projected concrete failure area of an anchor or group of anchors, for calculation of strength in shear, as defined in Section 1913.6.2.1, inches squared. A_V shall not be taken greater than nA_{Vo}.

c = Distance from center of an anchor shaft to the edge of concrete, inches.

c_l = Distance from the center of an anchor shaft to the edge of concrete in one direction, in. Where shear force is applied to anchor, c_l is in the direction of the shear force.

c_2 = Distance from center of an anchor shaft to the edge of concrete in the direction orthogonal to c_l, inches.

c_{max} = The largest of the edge distances that are less than or equal to 1.5 h_{ef}, inches (used only for the case of 3 or 4 edges).

c_{min} = The smallest of the edge distances that are less than or equal to 1.5 h_{ef}, inches.

d_o = Shaft diameter of headed stud, headed bolt, or hooked anchor, inches.

d_u = Diameter of head of stud or anchor bolt or equivalent diameter of effective perimeter of an added plate or washer at the head of the anchor, inches.

e_h = Distance from the inner surface of the shaft of a J-bolt or L-bolt to the outer tip of the J- or L-bolt, inches.

e_N' = Eccentricity of normal force on a group of anchors; the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension, inches.

e_V' = Eccentricity of shear force on a group of anchors; the distance between the point of shear force application and the centroid of the group of anchors resisting shear in the direction of the applied shear, inches.

f'_c = Compressive strength of concrete, pound per square inch.

f_{ct} = Specified tensile strength of concrete, psi.

f_r = Modulus of rupture of concrete, psi. (See Section 9.5.2.3 of ACI 318.)

f_t = Calculated tensile stress in a region of a member, psi.

f_y = Specified yield strength of anchor steel, psi.

f_{ut} = Specified tensile strength of anchor steel, psi.

h = Thickness of member in which an anchor is embedded measured parallel to anchor axis, inches.

h_{ef} = Effective anchor embedment depth, inches.

k = Coefficient for basic concrete breakout strength in tension.

k_{cp} = Coefficient for pryout strength.

l = Load-bearing length of anchor for shear, not to exceed $8d_o$, inches.

l = h_{ef} for anchors with a constant stiffness over the full length of the embedded section, such as headed studs.

n = Number of anchors in a group.

N_b = Basic concrete breakout strength in tension of a single anchor in cracked concrete, as defined in Section 1913.5.2.2, pounds.

N_{cb} = Nominal concrete breakout strength in tension of a single anchor, as defined in Section 1913.5.2.1, pounds.

N_{cbg} = Nominal concrete breakout strength in tension of a group of anchors, as defined in Section 1913.5.2.1, pounds.

N_n = Nominal strength in tension, pounds.

N_p = Pullout strength in tension of a single anchor in cracked concrete, as defined in Section 1913.5.3.3 or 1913.5.3.4, pounds.

N_{pn} = Nominal pullout strength in tension of a single anchor, as defined in Section 1913.5.3.1, pounds.

N_{sb} = Side-face blowout strength of a single anchor, pounds.

N_{sbg} = Side-face blowout strength of a group of anchors, pounds.

N_s = Nominal strength of a single anchor in tension as governed by the steel strength, as defined in Section 1913.5.1.2, pounds.

N_u = Factored tensile load, pounds.

s = Anchor center-to-center spacing, inches.

s_o = Spacing of the outer anchors along the edge in a group, inches.

t = Thickness of washer or plate, inches.

V_b = Basic concrete breakout strength in shear of a single anchor in cracked concrete, as defined in Section 1913.6.2.2 or 1913.6.2.3, pound.

V_{cb} = Nominal concrete breakout strength in shear of a single anchor, as defined in Section 1913.6.2.1, pound.

V_{cbg} = Nominal concrete breakout strength in shear of a group of anchors, as defined in Section 1913.6.2.1, pound.

V_{cp} = Nominal concrete pryout strength, as defined in Section 1913.6.3, pound.

V_n = Nominal shear strength, pound.

V_s = Nominal strength in shear of a single anchor as governed by the steel strength, as defined in Section 1913.6.1.1, pound.

V_u = Factored shear load, pound.

ϕ = Strength reduction factor (see Sections 1913.4.4 and 1913.4.5).

ψ_1 = Modification factor, for strength in tension, to account for anchor groups loaded eccentrically, as defined in Section 1913.5.2.4.

ψ_2 = Modification factor, for strength in tension, to account for edge distances smaller than $1.5h_{ef}$, as defined in Section 1913.5.2.5.

ψ_3 = Modification factor, for strength in tension, to account for cracking, as defined in Sections 1913.5.2.6 and 1913.5.2.7.

ψ_4 = Modification factor, for pullout strength, to account for cracking, as defined in Sections 1913.5.3.1 and 1913.5.3.5.

ψ_5 = Modification factor, for strength in shear, to account for anchor groups loaded eccentrically, as defined in Section 1913.6.2.5.

ψ_6 = Modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$, as defined in Section 1913.6.2.6.

ψ_7 = Modification factor, for strength in shear, to account for cracking, as defined in Section 1913.6.2.7.

1913.2.2 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

ANCHOR. A metallic element used to transmit applied loads including headed bolts, headed studs, and hooked bolts (J- or L-bolt).

ANCHOR GROUP. A number of anchors of approximately equal effective embedment depth with each anchor spaced at less than three times its embedment depth from one or more adjacent anchors.

ANCHOR PULLOUT STRENGTH. The strength corresponding to the fastening device sliding out from the concrete without breaking out a substantial portion of the surrounding concrete.

ATTACHMENT. The element or assembly, external to the surface of the concrete, that transmits loads to or from the anchor.

BRITTLE STEEL ELEMENT. An element with a tensile test elongation of less than 14 percent over a 2-inch (51 mm) gage length, reduction in area of less than 40 percent, or both.

CONCRETE BREAKOUT STRENGTH. The strength corresponding to a volume of concrete surrounding the anchor or group of anchors separating from the member.

CONCRETE PRYOUT STRENGTH. The strength corresponding to formation of a concrete spall behind a short, stiff anchor with an embedded base that is displaced in the direction opposite to the applied shear force.

DUCTILE STEEL ELEMENT. An element with a tensile test elongation of at least 14 percent over a 2-inch gage length and reduction in area of at least 40 percent.

EDGE DISTANCE. The distance from the edge of the concrete surface to the center of the nearest anchor.

EFFECTIVE EMBEDMENT DEPTH. The overall depth through which the anchor transfers force to the surrounding concrete. The effective embedment depth will normally be the depth of the failure surface in tension applications. For headed anchor bolts and headed studs, the effective embedment depth is measured from the bearing contact surface of the head.

5-PERCENT FRACTILE. A statistical term meaning 90 percent confidence that 95 percent of the actual strengths will exceed the nominal strength. Determination shall include the number of tests when evaluating data.

HOOKED BOLT. An anchor anchored mainly by mechanical interlock from the 90-degree (1.57 rad) bend (L-bolt) or 180-degree (3.14 rad) bend (J-bolt) at its embedded end.

PROJECTED AREA. The area on the free surface of the concrete member that is used to represent the larger base of the assumed rectilinear failure surface.

SIDE-FACE BLOWOUT STRENGTH. The strength of anchors with deeper embedment but thinner side cover corresponding to concrete spalling on the side face around the embedded head while no major breakout occurs at the top concrete surface.

1913.3 General requirements.

1913.3.1 Anchorage design. Anchors and anchor groups shall be designed for critical effects of factored loads as determined by elastic analysis. Plastic analysis approaches are permitted where nominal strength is controlled by ductile steel elements, provided that deformational compatibility is taken into account.

1913.3.2 Load combinations. Except for load combinations that include earthquake forces or effects, anchors shall be designed for the load combinations of Section 9.2 of ACI 318. Where resistance to specified earthquake loads or forces E are included in design, the load combinations of Section 1605.2 shall apply.

1913.3.3 Seismic requirements. When anchor design includes seismic loads, the following additional requirements shall apply.

1913.3.3.1 Design strength. In structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1616, the design strength of anchors shall be taken as $0.75\phi N_n$ and $0.75\phi V_n$, where ϕ is given in Section 1913.4.4 or 1913.4.5 and N_n and V_n are determined in accordance with Section 1913.4.1.

1913.3.3.2 Governing strength. In structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1616, anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless Section 1913.3.3.3 is satisfied.

1913.3.3.3 Ductile yielding. In lieu of Section 1913.3.3.2, the attachment shall be designed so that it will undergo ductile yielding at a load level no greater than 75 percent of the minimum anchor design strength.

1913.3.4 Lightweight aggregate concrete modifications. All provisions for anchor axial tension and shear strength apply to normal weight concrete. When lightweight aggregate concrete is used, provisions for N_n and V_n shall be modified by multiplying all values of $\sqrt{f'_c}$ affecting N_n and V_n by 0.75 for "all-lightweight" concrete and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be permitted when partial sand replacement is used.

1913.3.5 Maximum concrete strength. The value of f'_c used for calculations in this section shall not exceed 10,000 psi (68.9 MPa).

1913.4 General requirements for strength of anchors.

1913.4.1 Strength considerations. Strength design of anchors shall be based on the computation or test that takes all of the following into consideration:

1. Steel strength of anchor in tension (Section 1913.5.1).
2. Steel strength of anchor in shear (Section 1913.6.1).
3. Concrete breakout strength of anchor in tension (Sections 1913.4.2 and 1913.5.2).
4. Concrete breakout strength of anchor in shear (Sections 1913.4.2 and 1913.6.2).
5. Pullout strength of anchor in tension (Sections 1913.4.2 and 1913.5.3).

6. Concrete side-face blowout strength of anchor in tension (Sections 1913.4.2 and 1913.5.4).
7. Concrete pryout strength of anchor in shear (Sections 1913.4.2 and 1913.6.3).
8. Required edge distances, spacings and thicknesses to preclude splitting failure (Sections 1913.4.2 and 1913.8).

1913.4.1.1 Strength limits. The design strengths of anchors in tension or shear, except as required in Section 1913.3.3, shall satisfy:

$$\phi N_n \geq N_u \qquad \textbf{(Equation 19-3)}$$

$$\phi V_n \geq V_u \qquad \textbf{(Equation 19-4)}$$

1913.4.1.2 Interaction effects. When both N_u and V_u are present, interaction effects shall be considered in accordance with Section 1913.4.3.

1913.4.1.3 Definitions. In Equations 19-3 and 19-4, ϕN_n and ϕV_n are the lowest design strengths determined from all appropriate failure modes. ϕN_n is the lowest design strength in tension of an anchor or group of anchors as determined from consideration of ϕN_s, ϕN_{pn}, either ϕN_{sb} or ϕN_{sbg}, and either ϕN_{cb} or ϕN_{cbg}. ϕV_n is the lowest design strength in shear of an anchor or a group of anchors as determined from consideration of ϕV_s, either ϕV_{cb} or ϕV_{cbg}, and ϕV_{cp}.

1913.4.2 Design models. The nominal strength for any anchor or group of anchors shall be based on design models that result in predictions of strength in substantial agreement with results of comprehensive tests. The materials used in the tests shall be compatible with the materials to be used in the structure. The nominal strength shall be based on the 5-percent fractile of the basic individual anchor strength, with modifications made for the number of anchors, the effects of close spacing of anchors, proximity to edges, depth of the concrete member, eccentric loadings of anchor groups, and presence or absence of cracking. Limits on edge distances and anchor spacing in the design models shall be consistent with the tests that verified the model.

1913.4.2.1 Supplementary reinforcements. The effect of supplementary reinforcement provided to confine or restrain the concrete breakout, or both, shall be permitted to be included in the design models of Section 1913.4.2.

1913.4.2.2 Breakout strength. For anchors with diameters not exceeding 2 inches (51 mm), and tensile embedments not exceeding 25 inches (635 mm) in depth, the concrete breakout strength requirements of Section 1913.4.2 shall be considered satisfied by the design procedure of Sections 1913.5.2 and 1913.6.2.

1913.4.3 Combined loads. Resistance to combined tensile and shear loads shall be considered in design using an interaction expression that results in computation of strength in substantial agreement with results of comprehensive tests. This requirement shall be considered satisfied by Section 1913.7.

1913.4.4 Reduction factors. Strength reduction factor ϕ for fastening to concrete shall be as follows when the load combinations of Section 9.2 of ACI 318 or Section 1605.2 of this code are used:

1. Anchor governed by tensile or shear strength of a ductile steel element, 0.90.
2. Anchor governed by tensile or shear strength of a brittle steel element, 0.75.
3. Anchor governed by concrete breakout, blowout, pullout or pryout strength.

	Condition A	Condition B
3.1. Shear Loads	0.85	0.75
3.2. Tension Loads	0.85	0.75

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

1913.4.5 Alternate reduction factors. Strength reduction factor ϕ for fastening to concrete shall be as follows when the load combinations referenced in Appendix C of ACI 318 are used:

1. Anchor governed by tensile or shear strength of a ductile steel element, 0.80.
2. Anchor governed by tensile or shear strength of a brittle steel element, 0.70.
3. Anchor governed by concrete breakout, blowout, pullout or pryout strength.

	Condition A	Condition B
3.1. Shear Loads	0.75	0.70
3.2. Tension Loads	0.75	0.70

Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member.

Condition B applies where such supplementary reinforcement is not provided or where pullout or pryout strength governs.

1913.5 Design requirements for tensile loading.

1913.5.1 Steel strength of anchor in tension.

1913.5.1.1 Strength determination options. The nominal strength of an anchor in tension as governed by the steel, N_s, shall be determined by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Alternatively, it shall be permitted to use values based on the 5-percent fractile of test results to establish values of N_s.

1913.5.1.2 Calculated strength. Unless determined by the 5-percent fractile of test results, nominal strength of an anchor or group of anchors in tension shall not exceed:

1. For anchor material with a well-defined yield point:

$N_s = nA_{se}f_y$ **(Equation 19-5)**

2. For anchor material without a well-defined yield point where f_{ut} shall not be taken greater than 125,000 psi (861.9 MPa):

$N_s = nA_{se}(0.8f_{ut})$ **(Equation 19-6)**

1913.5.2 Concrete breakout strength of anchor in tension.

1913.5.2.1 Nominal breakout strength. Unless determined in accordance with Section 1913.4.2, nominal concrete breakout strength of an anchor or group of anchors in tension shall not exceed:

1. For an anchor:

$$N_{cb} = \frac{A_N}{A_{No}}\psi_2\psi_3 N_b \qquad \textbf{(Equation 19-7)}$$

2. For a group of anchors:

$$N_{cbg} = \frac{A_N}{A_{No}}\psi_1\psi_2\psi_3 N_b \qquad \textbf{(Equation 19-8)}$$

N_b is the basic concrete breakout strength value for a single anchor in tension in cracked concrete. A_N is the projected area of the failure surface for the anchor or group of anchors that shall be approximated as the base of the rectilinear geometrical figure that results from projecting the failure surface outward $1.5h_{ef}$ from the centerlines of the anchor, or in the case of a group of anchors, from a line through a row of adjacent anchors. A_N shall not exceed nA_{No}, where n is the number of tensioned anchors in the group. A_{No} is the projected area of the failure surface of a single anchor remote from edges:

$$A_{No} = 9h_{ef}^2 \qquad \textbf{(Equation 19-9)}$$

1913.5.2.2 Basic breakout strength. Unless determined in accordance with Section 1913.4.2, the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed:

$$N_b = k\sqrt{f'_c}\,h_{ef}^{1.5} \qquad \textbf{(Equation 19-10)}$$

where k =24.

Alternatively, for headed studs and headed bolts with 11 inches $< h_{ef} <$ 25 inches, the basic concrete breakout strength of a single anchor in tension in cracked concrete shall not exceed:

$$N_b = k\sqrt{f'_c}\,h_{ef}^{5/3} \qquad \textbf{(Equation 19-11)}$$

where k =16.

1913.5.2.3 Limited edge distance. For the special case of anchors in an application with three or four edges and the largest edge distance $c_{max} < 1.5\ h_{ef}$, the embedment depth h_{ef} used in Equations 19-9, 19-10, 19-11, 19-12, 19-13 and 19-14 shall be limited to $c_{max}/1.5$.

1913.5.2.4 Eccentric loading. The modification factor for eccentrically loaded anchor groups where $e_N' < s/2$ is:

$$\psi_1 = \frac{1}{\left(1+\dfrac{2e_{N'}}{3h_{ef}}\right)} \le 1 \qquad \textbf{(Equation 19-12)}$$

If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered when determining the eccentricity, e_N', for use in Equation 19-12.

In the case where eccentric loading exists about two axes, the modification factor, ψ_1, shall be computed for each axis individually and the product of these factors used as ψ_1 in Equation 19-8.

1913.5.2.5 Edge effects. The modification factor for edge effects is:

$$\psi_2 = 1 \text{ if } c_{min} \ge 1.5h_{ef} \qquad \textbf{(Equation 19-13)}$$

$$\psi_2 = 0.7 + 03\frac{c_{min}}{1.5h_{ef}} \text{ if } c_{min} < 1.5h_{ef} \qquad \textbf{(Equation 19-14)}$$

1913.5.2.6 No cracking at service load. Where an anchor is located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor is permitted:

$$\psi_3 = 1.25 \qquad \textbf{(Equation 19-15)}$$

1913.5.2.7 Cracking at service load. When analysis indicates cracking at service load levels, ψ_3 shall be taken as 1.0. The cracking in the concrete shall be controlled by flexural reinforcement distributed in accordance with Section 10.6.4 of ACI 318, or equivalent crack control shall be provided by confining reinforcement.

1913.5.2.8 Added plate or washer. Where an additional plate or washer is added under the head of the anchor, it shall be permitted to calculate the projected area of the failure surface by projecting the failure surface outward $1.5h_{ef}$ from the effective perimeter of the plate or washer. The effective perimeter shall not exceed the value at a section projected outward more than t from the outer edge of the head of anchor, where t is the thickness of the washer or plate.

1913.5.3 Pullout strength of anchor in tension.

1913.5.3.1 Nominal pullout strength. Unless determined in accordance with Section 1913.4.2, the nominal pullout strength of an anchor in tension shall not exceed:

$$N_{pn} = \psi_4 N_p \qquad \textbf{(Equation 19-16)}$$

1913.5.3.2 Stud and bolt type. For single-headed studs and headed bolts, it is permitted to calculate the pullout strength in tension using Section 1913.5.3.3. For single J-bolts or L-bolts, it is permitted to calculate the pullout strength in tension using Section 1913.5.3.4. Alternatively, it is permitted to use values of N_p based on the 5-percent fractile of tensile tests performed in such a manner as to exclude the benefit of friction.

1913.5.3.3 Headed stud or bolt. Unless determined in accordance with Section 1913.4.2, the pullout strength in tension of a single-headed stud or headed bolt, N_p for use in Equation 19-16, shall not exceed:

$$N_p = A_b 8f'_c \qquad \textbf{(Equation 19-17)}$$

1913.5.3.4 J-bolt or L-bolt. Unless determined in accordance with Section 1913.4.2, the pullout strength in tension of a single J-bolt or L-bolt, N_p for use in Equation 19-16, shall not exceed:

$$N_p = 0.9f'_c e_h d_o \qquad \textbf{(Equation 19-18)}$$

where: $3d_o < e_h < 4.5d_o$.

1913.5.3.5 No cracking at service load. For an anchor located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service load levels, the following modification factor is permitted:

$$\psi_4 = 1.4 \qquad \textbf{(Equation 19-19)}$$

Otherwise, ψ_4 shall be taken as 1.0.

1913.5.4 Concrete side-face blowout strength of a headed anchor in tension.

1913.5.4.1 Single headed anchor. For a single headed anchor with deep embedment close to an edge ($c < 0.4h_{ef}$), unless determined in accordance with Section 1913.4.2, the nominal side-face blowout strength N_{sb} shall not exceed:

$$N_{sb} = 160c\sqrt{A_b}\sqrt{f'_c} \qquad \textbf{(Equation 19-20)}$$

If the single headed anchor is located at a perpendicular distance c_2 less than $3c$ from an edge, the value of N_{sb} shall be modified by multiplying it by the factor $(1 + c_2/c)/4$ where $1 < c_2/c < 3$.

1913.5.4.2 Group of headed anchors. For a group of headed anchors with deep embedment close to an edge ($c < 0.4\ h_{ef}$) and spacing between anchors less than $6c$, unless determined in accordance with Section 1913.4.2, the nominal strength of the group of anchors for a side-face blowout failure N_{sbg} shall not exceed:

$$N_{sbg} = \left(1 + \frac{s_o}{6c}\right) N_{sb} \qquad \textbf{(Equation 19-21)}$$

where s_o is the spacing of the outer anchors along the edge in the group and N_{sb} is obtained from Equation 19-20 without modification for a perpendicular edge distance.

1913.6 Design requirements for shear loading.

1913.6.1 Steel strength of anchor in shear.

1913.6.1.1 General. The nominal strength of an anchor in shear as governed by steel, V_s, shall be determined by calculations based on the properties of the anchor material and the physical dimensions of the anchor. Alternatively, it shall be permitted to use values based on the 5-percent fractile of test results to establish values of V_s.

1913.6.1.2 Calculated strength. Unless determined by the 5-percent fractile of test results, nominal strength of an anchor or group of anchors in shear shall not exceed:

1. For anchors with a well-defined yield point:

$$V_s = nA_{se}f_y \qquad \textbf{(Equation 19-22)}$$

2. For anchors without a well-defined yield point:

$$V_s = n0.6A_{se}f_{ut} \qquad \textbf{(Equation 19-23)}$$

where f_{ut} shall not be taken greater than 125,000 psi (861 MPa).

1913.6.1.3 Anchors with grout pads. Where anchors are used with built-up grout pads, the nominal strengths of Section 1913.6.1.2 shall be reduced by 20 percent.

1913.6.2 Concrete breakout strength of anchor in shear.

1913.6.2.1 Nominal shear strength. Unless determined in accordance with Section 1913.4.2, nominal concrete breakout strength in shear of an anchor or group of anchors shall not exceed:

1. For shear force perpendicular to the edge on a single anchor:

$$V_{cb} = \frac{A_v}{Av_o}\psi_6\psi_7 V_b \qquad \textbf{(Equation 19-24)}$$

2. For shear force perpendicular to the edge on a group of anchors:

$$V_{cbg} = \frac{A_v}{Av_o}\psi_5\psi_6\psi_7 V_b \qquad \textbf{(Equation 19-25)}$$

3. For shear force parallel to an edge, V_{cb} or V_{cbg} shall be permitted to be twice the value for shear force determined from Equations 19-24 and 19-25, respectively, with ψ_6 taken equal to 1.
4. For anchors located at a corner, the limiting nominal concrete breakout strength shall be determined for each edge and the minimum value shall be used.

V_b is the basic concrete breakout strength value for a single anchor. A_V is the projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. It shall be permitted to evaluate this area as the base of a truncated half pyramid projected on the side face of the member where the top of the half pyramid is given by the axis of the anchor row selected as critical. The value of c_1 shall be taken as the distance from the edge to this axis. A_V shall not exceed nA_{Vo}, where n is the number of anchors in the group.

A_{Vo} is the projected area for a single anchor in a deep member and remote from edges in the direction perpendicular to the shear force. It shall be permitted to evaluate this area as the base of a half pyramid with a side length parallel to the edge of $3c_1$ and a depth of $1.5c_1$:

$$Av_o = 4.5c_1^2 \qquad \textbf{(Equation 19-26)}$$

Where anchors are located at varying distances from the edge and the anchors are welded to the attachment so as to distribute the force to all anchors, it shall be permitted to evaluate the strength based on the distance to the farthest row of anchors from the edge. In this case, it shall be permitted to base the value of c_1 on the distance from the edge to the axis of the farthest anchor row which is selected as critical, and all of the shear shall be assumed to be carried by this critical anchor row alone.

1913.6.2.2 Basic concrete breakout strength. Unless determined in accordance with Section 1913.4.2, the basic concrete breakout strength in shear of a single anchor in cracked concrete shall not exceed:

$$V_b = 7\left[\frac{\ell}{d_o}\right]^{0.2}\sqrt{d_o}\sqrt{f'_c}\,c_1^{1.5} \qquad \textbf{(Equation 19-27)}$$

1913.6.2.3 Welded attachments. For anchors that are rigidly welded to steel attachments having a minimum thickness equal to the greater of $^3/_8$ inch (9.5 mm) or half of the anchor diameter, unless determined in accordance with Section 1913.4.2, the basic concrete breakout strength in shear of a single anchor in cracked concrete shall not exceed:

$$V_b = 8\left[\frac{\ell}{d_o}\right]^{0.2}\sqrt{d_o}\sqrt{f'_c}\,c_1^{1.5} \qquad \textbf{(Equation 19-28)}$$

provided that:

1. For groups of anchors, the strength is determined based on the strength of the row of anchors farthest from the edge.
2. The center-to-center spacing of the anchors is not less than 2.5 inch (63.5 mm).
3. Supplementary reinforcement is provided at the corners if $c_2 \leq 1.5h_{ef}$.

1913.6.2.4 Anchors in thin members. For the special case of anchors in a thin member influenced by three or more edges, the edge distance c_1 used in Equations 19-26, 19-27, 19-28, 19-29, 19-30 and 19-31 shall be limited to $h/1.5$.

1913.6.2.5 Eccentric loading. The modification factor for eccentrically loaded anchor groups where $e'_v \leq s/2$ is:

$$\psi_5 = \frac{1}{1+\dfrac{2e'_v}{3c_1}} \leq 1 \qquad \textbf{(Equation 19-29)}$$

1913.6.2.6 Edge effects. The modification factor for edge effects is:

$$\psi_6 = 1 \text{ if } c_2 \geq 1.5c_1 \qquad \textbf{(Equation 19-30)}$$

$$\psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1} \text{ if } c_2 < 1.5c_1 \qquad \textbf{(Equation 19-31)}$$

1913.6.2.7 No cracking at service load. For anchors located in a region of a concrete member where analysis indicates no cracking ($f_t < f_r$) at service loads, the following modification factor is permitted:

$$\psi_7 = 1.4 \qquad \textbf{(Equation 19-32)}$$

1913.6.2.8 Cracking at service load. For anchors located in a region of a concrete member where analysis indicates cracking at service load levels, the following modification factors shall be permitted. In order to be considered as edge reinforcement, the reinforcement shall be designed to intersect the concrete breakout:

$\psi_7 =$ 1.0 for anchors in cracked concrete with no edge reinforcement or edge reinforcement smaller than a No. 4 bar.

$\psi_7 =$ 1.2 for anchors in cracked concrete with edge reinforcement of a No. 4 bar or greater between the anchor and the edge.

$\psi_7 =$ 1.4 for anchors in cracked concrete with edge reinforcement of a No. 4 bar or greater between the anchor and the edge and with the edge reinforcement enclosed within stirrups spaced at not more than 4 inches (102 mm).

1913.6.3 Concrete pryout strength of anchor in shear. Unless determined in accordance with Section 1913.4.2, the nominal pryout strength, V_{cp}, shall not exceed:

$$V_{cp} = k_{cp}N_{cb} \qquad \textbf{(Equation 19-33)}$$

where:

k_{cp} = 1.0 for $h_{ef} < 2.5$ inches (63.5 mm)

k_{cp} = 2.0 for $h_{ef} > 2.5$ inches (63.5 mm)

and N_{cb} shall be determined from Equation 19-7.

1913.7 Interaction of tensile and shear forces. Unless determined in accordance with Section 1913.4.3, anchors or groups of anchors that are subjected to both shear and tension shall be designed to satisfy the following requirements:

1. If $V_u \leq 0.2\,\phi\,V_n$, then full design strength in tension is permitted: $\phi N_n \geq N_u$.
2. If $N_u \leq 0.2\,\phi\,N_n$, then full design strength in shear is permitted: $\phi\,V_n \geq V_u$.
3. If $V_u > 0.2\,\phi\,V_n$, and $N_u > 0.2\,\phi N_n$, then:

$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \leq 1.2 \qquad \textbf{(Equation 19-34)}$$

The value of ϕN_n shall be the smallest of the steel strength of the anchor in tension, concrete breakout strength of anchor in tension, pullout strength of anchor in tension, and side-face blowout strength. The value of ϕV_n shall be the smallest of the steel strength of anchor in shear, the concrete breakout strength of anchor in shear, and the pryout strength.

1913.8 Required edge distances, and spacings to preclude splitting failure. Minimum spacings and edge distances for anchors shall conform to Sections 1913.8.1 and 1913.8.2, unless reinforcement is provided to control splitting.

1913.8.1 Torqued and untorqued anchors. Unless determined in accordance with Section 1913.8.2, minimum edge distances for headed anchors that will not be torqued shall be based on minimum cover requirements for reinforcement in accordance with Section 7.7 of ACI 318. For headed anchors that will be torqued, the minimum edge distances shall be $6d_o$.

1913.8.2 Limited edge distance. For anchors that will remain untorqued, if the edge distance or spacing is less than that specified in Section 1913.8.1, calculations shall be performed using a fictitious value of d_o that meets the requirements of Section 1913.8.1. Calculated forces applied to the anchor shall be limited to the values corresponding to an anchor having that fictitious diameter.

1913.8.3 Construction documents. Construction documents shall specify use of anchors with a minimum edge distance as assumed in design.

1913.9 Installation of anchors. Anchors shall be installed in accordance with the construction documents.

SECTION 1914
SHOTCRETE

1914.1 General. Shotcrete is mortar or concrete that is pneumatically projected at high velocity onto a surface. Except as specified in this section, shotcrete shall conform to the requirements of this chapter for plain or reinforced concrete.

1914.2 Proportions and materials. Shotcrete proportions shall be selected that allow suitable placement procedures using the delivery equipment selected and shall result in finished in-place hardened shotcrete meeting the strength requirements of this code.

1914.3 Aggregate. Coarse aggregate, if used, shall not exceed $^3/_4$ inch (19.1 mm).

1914.4 Reinforcement. Reinforcement used in shotcrete construction shall comply with the provisions of Sections 1914.4.1 through 1914.4.4.

1914.4.1 Size. The maximum size of reinforcement shall be No. 5 bars unless it is demonstrated by preconstruction tests that adequate encasement of larger bars will be achieved.

1914.4.2 Clearance. When No. 5 or smaller bars are used, there shall be a minimum clearance between parallel reinforcement bars of $2^1/_2$ inches (64 mm). When bars larger than No. 5 are permitted, there shall be a minimum clearance between parallel bars equal to 6 diameters of the bars used. When two curtains of steel are provided, the curtain nearer the nozzle shall have a minimum spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of 6 bar diameters.

Exception: Deleted.

1914.4.3 Splices. Lap splices of reinforcing bars shall utilize the noncontact lap splice method with a minimum clearance of 2 inches (51 mm) between bars. The use of contact lap splices necessary for support of the reinforcing is permitted when approved by the building official, based on satisfactory preconstruction tests that show that adequate encasement of the bars will be achieved, and provided that the splice is oriented so that a plane through the center of the spliced bars is perpendicular to the surface of the shotcrete.

1914.4.4 Spirally tied columns. Shotcrete shall not be applied to spirally tied columns.

1914.5 Preconstruction tests. When required by the building official, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design. It shall be shot at the same angle, using the same nozzleman and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is approved by the building official.

1914.6 Rebound. Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be used as aggregate.

1914.7 Joints. Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the approved construction documents, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

1914.8 Damage. In-place shotcrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while still plastic.

1914.9 Curing. During the curing periods specified herein, shotcrete shall be maintained above 40°F (4°C) and in moist condition.

1914.9.1 Initial curing. Shotcrete shall be kept continuously moist for 24 hours after shotcreting is complete or shall be sealed with an approved curing compound.

1914.9.2 Final curing. Final curing shall continue for seven days after shotcreting, or for three days if high-early-strength cement is used, or until the specified strength is obtained. Final curing shall consist of the initial curing process or the shotcrete shall be covered with an approved moisture-retaining cover.

1914.9.3 Natural curing. Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the registered design professional and approved by the building official.

1914.10 Strength tests. Strength tests for shotcrete shall be made by an approved agency on specimens that are representative of the work and which have been water soaked for at least 24 hours prior to testing. When the maximum size aggregate is larger than $^3/_8$ inch (9.5 mm), specimens shall consist of not less than three 3-inch (76 mm) diameter cores or 3-inch (76 mm) cubes. When the maximum size aggregate is $^3/_8$ inch (9.5 mm) or smaller, specimens shall consist of not less than 2-inch (51 mm) diameter cores or 2-inch (51 mm) cubes.

1914.10.1 Sampling. Specimens shall be taken from the in-place work or from test panels, and shall be taken at least once each shift, but not less than one for each 50 cubic yards (38.2 m^3) of shotcrete.

1914.10.2 Panel criteria. When the maximum size aggregate is larger than $^3/_8$ inch (9.5 mm), the test panels shall have minimum dimensions of 18 inches by 18 inches (457 mm by 457 mm). When the maximum size aggregate is $^3/_8$ inch (9.5 mm) or smaller, the test panels shall have minimum dimensions of 12 inches by 12 inches (305 mm by 305 mm). Panels shall be shot in the same position as the work, during the course of the work and by the nozzlemen doing the work. The conditions under which the panels are cured shall be the same as the work.

1914.10.3 Acceptance criteria. The average compressive strength of three cores from the in-place work or a single test panel shall equal or exceed $0.85f'_c$ with no single core less than $0.75\,f'_c$. The average compressive strength of three cubes taken from the in-place work or a single test panel shall equal or exceed f'_c with no individual cube less than $0.88f'_c$. To check accuracy, locations represented by erratic core or cube strengths shall be retested.

SECTION 1915 REINFORCED GYPSUM CONCRETE

1915.1 General. Reinforced gypsum concrete shall comply with the requirements of ASTM C 317 and ASTM C 956.

1915.2 Minimum thickness. The minimum thickness of reinforced gypsum concrete shall be 2 inches (51 mm) except the minimum required thickness shall be reduced to $1^1/_2$ inches (38 mm), provided the following conditions are satisfied:

1. The overall thickness, including the formboard, is not less than 2 inches (51 mm).
2. The clear span of the gypsum concrete between supports does not exceed 33 inches (838 mm).
3. Diaphragm action is not required.
4. The design live load does not exceed 40 psf (1915 Pa).

SECTION 1916 CONCRETE-FILLED PIPE COLUMNS

1916.1 General. Concrete-filled pipe columns shall be manufactured from standard, extra-strong or double-extra-strong steel pipe or tubing that is filled with concrete so placed and manipulated as to secure maximum density and to ensure complete filling of the pipe without voids.

1916.2 Design. The safe supporting capacity of concrete-filled pipe columns shall be computed in accordance with the approved rules or as determined by a test.

1916.3 Connections. Caps, base plates and connections shall be of approved types and shall be positively attached to the shell and anchored to the concrete core. Welding of brackets without mechanical anchorage shall be prohibited. Where the pipe is slotted to accommodate webs of brackets or other connections, the integrity of the shell shall be restored by welding to ensure hooping action of the composite section.

1916.4 Reinforcement. To increase the safe load-supporting capacity of concrete-filled pipe columns, the steel reinforcement shall be in the form of rods, structural shapes or pipe embedded in the concrete core with sufficient clearance to ensure the composite action of the section, but not nearer than 1 inch (25 mm) to the exterior steel shell. Structural shapes used as reinforcement shall be milled to ensure bearing on cap and base plates.

1916.5 Fire-resistance-rating protection. Pipe columns shall be of such size or so protected as to develop the required fire-resistance ratings specified in Table 601. Where an outer steel shell is used to enclose the fire-resistive covering, the shell shall not be included in the calculations for strength of the column section. The minimum diameter of pipe columns shall be 4 inches (102 mm) except that in structures of Type V construction not exceeding three stories or 40 feet (12 192 mm) in height, pipe columns used in the basement and as secondary steel members shall have a minimum diameter of 3 inches (76 mm).

1916.6 [Comm 62.1916] Approvals. Details of column connections and splices shall be shop-fabricated by approved methods and testing. Shop-fabricated concrete-filled pipe columns shall be inspected by the building official or by a representative of the manufacturer at the plant.

CHAPTER 20
ALUMINUM

SECTION 2001
GENERAL

2001.1 Scope. This chapter shall govern the quality, design, fabrication and erection of aluminum.

SECTION 2002
MATERIALS

2002.1 General. Aluminum used for structural purposes in buildings and structures shall comply with AA ASM 35 and Parts 1-A and 1-B of the Aluminum Design Manual. The nominal loads shall be the minimum design loads required by Chapter 16.

CHAPTER 21
MASONRY

SECTION 2101
GENERAL

2101.1 Scope. This chapter shall govern the materials, design, construction and quality of masonry.

2101.2 Design methods. Masonry shall comply with the provisions of one of the following design methods in this chapter as well as the requirements of Sections 2101 through 2105.

2101.2.1 Working stress design. Masonry designed by the working stress design method shall comply with the provisions of Sections 2106 and 2107.

2101.2.2 Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108.

2101.2.3 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Section 2106 and 2109 or Chapters 1 and 5 of 530/ASCE 5/TMS 402.

2101.2.4 Glass masonry. Glass masonry shall comply with the provisions of Section 2110 or with the requirements of Chapter 7 of 530/ASCE 5/TMS 402.

2101.2.5 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14.

2101.3 Deleted.

2101.3.1 Deleted.

SECTION 2102
DEFINITIONS AND NOTATIONS

2102.1 General. The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meanings shown herein.

ADOBE CONSTRUCTION. Construction in which the exterior bearing and nonbearing walls and partitions are of unfired clay masonry units, and floors, roofs and interior framing are wholly or partly of wood or other approved materials.

Adobe, stabilized. Unfired clay masonry units to which admixtures, such as emulsified asphalt, are added during the manufacturing process to limit the units' water absorption so as to increase their durability.

Adobe, unstabilized. Unfired clay masonry units that do not meet the definition of adobe, stabilized.

ANCHOR. Metal rod, wire or strap that secures masonry to its structural support.

ARCHITECTURAL TERRA COTTA. Plain or ornamental hard-burned modified clay units, larger in size than brick, with glazed or unglazed ceramic finish.

AREA

Bedded. The area of the surface of a masonry unit that is in contact with mortar in the plane of the joint.

Gross cross-sectional. The area delineated by the out-to-out specified dimensions of masonry in the plane under consideration.

Net cross-sectional. The area of masonry units, grout and mortar crossed by the plane under consideration based on out-to-out specified dimensions.

BED JOINT. The horizontal layer of mortar on which a masonry unit is laid.

BOND BEAM. A horizontal grouted element within masonry in which reinforcement is embedded.

BOND REINFORCING. The adhesion between steel reinforcement and mortar or grout.

BRICK

Calcium silicate (sand lime brick). A building unit made of sand and lime.

Clay or shale. A masonry unit made of clay or shale, usually formed into a rectangular prism while in the plastic state and burned or fired in a kiln.

Concrete. A masonry unit having the approximate shape of a rectangular prism and composed of inert aggregate particles embedded in a hardened cementitious matrix.

BUTTRESS. A projecting part of a masonry wall built integrally therewith to provide lateral stability.

CAST STONE. A building stone manufactured from portland cement concrete precast and used as a trim, veneer or facing on or in buildings or structures.

CELL. A void space having a gross cross-sectional area greater than $1^1/_2$ square inches (967 mm^2).

CHIMNEY. A primarily vertical enclosure containing one or more passageways for conveying flue gases to the outside atmosphere.

CHIMNEY TYPES

High-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, high-heat appliances producing combustion gases in excess of 2,000°F (1093°C) measured at the appliance flue outlet (see Section 2113.11.3).

Low-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, low-heat appliances producing combustion gases not in excess of 1,000°F (538°C) under normal operating conditions, but capable of producing combustion gases of 1,400°F (760°C) during intermittent forces firing for periods up to 1 hour. Temperatures shall be measured at the appliance flue outlet.

Masonry type. A field-constructed chimney of solid masonry units or stones.

Medium-heat appliance type. An approved chimney for removing the products of combustion from fuel-burning, medium-heat appliances producing combustion gases not exceeding 2,000°F (1093°C) measured at the appliance flue outlet (see Section 2113.11.2).

CLEANOUT. An opening to the bottom of a grout space of sufficient size and spacing to allow the removal of debris.

COLLAR JOINT. Vertical longitudinal joint between wythes of masonry or between masonry and back-up construction that is permitted to be filled with mortar or grout.

COLUMN, MASONRY. An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is at least three times its thickness.

COMPOSITE MASONRY. Multiwythe masonry members acting with composite action.

COMPRESSIVE STRENGTH OF MASONRY. Maximum compressive force resisted per unit of net cross-sectional area of masonry, determined by the testing of masonry prisms or a function of individual masonry units, mortar and grout.

CONNECTOR. A mechanical device for securing two or more pieces, parts or members together, including anchors, wall ties and fasteners.

COVER. Distance between surface of reinforcing bar and edge of member.

DIAPHRAGM. A roof or floor system designed to transmit lateral forces to shear walls or other lateral load-resisting elements.

DIMENSIONS

Actual. The measured dimension of a masonry unit or element.

Nominal. A dimension equal to a specified dimension plus an allowance for the joints with which the units are to be laid. Thickness is given first, followed by height and then length.

Specified. The dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other component of a structure.

EFFECTIVE HEIGHT. For braced members, the effective height is the clear height between lateral supports and is used for calculating the slenderness ratio. The effective height for unbraced members is calculated in accordance with engineering mechanics.

EFFECTIVE PERIOD. Fundamental period of the structure based on cracked stiffness.

FIREPLACE. A hearth and fire chamber or similar prepared place in which a fire may be made and which is built in conjunction with a chimney.

FIREPLACE THROAT. The opening between the top of the firebox and the smoke chamber.

GROUTED MASONRY

Grouted hollow-unit masonry. That form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

Grouted multiwythe masonry. That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

HEAD JOINT. Vertical mortar joint placed between masonry units within the wythe at the time the masonry units are laid.

HEADER (Bonder). A masonry unit that connects two or more adjacent wythes of masonry.

HEIGHT, WALLS. The vertical distance from the foundation wall or other immediate support of such wall to the top of the wall.

MASONRY. A built-up construction or combination of building units or materials of clay, shale, concrete, glass, gypsum, stone or other approved units bonded together with or without mortar or grout or other accepted method of joining.

Ashlar masonry. Masonry composed of various sized, rectangular units having sawed, dressed or squared bed surfaces, properly bonded and laid in mortar.

Coursed ashlar. Ashlar masonry laid in courses of stone of equal height for each course, although different courses shall be permitted to be of varying height.

Glass unit masonry. Nonload-bearing masonry composed of glass units bonded by mortar.

Plain masonry. Masonry in which the tensile resistance of the masonry is taken into consideration and the effects of stresses in reinforcement are neglected.

Random ashlar. Ashlar masonry laid in courses of stone set without continuous joints and laid up without drawn patterns. When composed of material cut into modular heights, discontinuous but aligned horizontal joints are discernible.

Reinforced masonry. Masonry construction in which reinforcement acting in conjunction with the masonry is used to resist forces.

Solid masonry. Masonry consisting of solid masonry units laid contiguously with the joints between the units filled with mortar.

MASONRY UNIT. Brick, tile, stone, glass block or concrete block conforming to the requirements specified in Section 2103.

Clay. A building unit larger in size than a brick, composed of burned clay, shale, fire clay or mixtures thereof.

Concrete. A building unit or block larger in size than 12 by 4 by 4 inches (305 mm by 102 mm by 102 mm) made of cement and suitable aggregates.

Hollow. A masonry unit whose net cross-sectional area in any plane parallel to the load-bearing surface is less than 75 percent of its gross cross-sectional area measured in the same plane.

Solid. A masonry unit whose net cross-sectional area in every plane parallel to the load-bearing surface is 75 percent or more of its gross cross-sectional area measured in the same plane.

MEAN DAILY TEMPERATURE. The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.

MORTAR. A plastic mixture of approved cementitious materials, fine aggregates and water used to bond masonry or other structural units.

MORTAR, SURFACE-BONDING. A mixture to bond concrete masonry units that contains hydraulic cement, glass fiber reinforcement with or without inorganic fillers or organic modifiers, and water.

PLASTIC HINGE. The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquakes.

PRISM. An assemblage of masonry units and mortar with or without grout used as a test specimen for determining properties of the masonry.

REQUIRED STRENGTH. Strength of a member or cross section required to resist factored loads.

RUBBLE MASONRY. Masonry composed of roughly shaped stones.

Coursed rubble. Masonry composed of roughly shaped stones fitting approximately on level beds and well bonded.

Random rubble. Masonry composed of roughly shaped stones laid without regularity of coursing but well bonded and fitted together to form well-divided joints.

Rough or ordinary rubble. Masonry composed of unsquared field stones laid without regularity of coursing but well bonded.

RUNNING BOND. The placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

SHEAR WALL

Detailed plain masonry shear wall. A masonry shear wall designed to resist lateral forces neglecting stresses in reinforcement, and designed in accordance with Section 2106.1.1.3.

Intermediate reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with Section 2106.1.1.4.

Ordinary plain masonry shear wall. A masonry shear wall designed to resist lateral forces neglecting stresses in reinforcement, and designed in accordance with Section 2106.1.1.1.

Ordinary reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with Section 2106.1.1.2.

Special reinforced masonry shear wall. A masonry shear wall designed to resist lateral forces considering stresses in reinforcement, and designed in accordance with Section 2106.1.1.5.

SHELL. The outer portion of a hollow masonry unit as placed in masonry.

SPECIFIED. Required by construction documents.

SPECIFIED COMPRESSIVE STRENGTH OF MASONRY, f'_m. Minimum compressive strength, expressed as force per unit of net cross-sectional area, required of the masonry used in construction by the construction documents, and upon which the project design is based. Whenever the quantity f'_m is under the radical sign, the square root of numerical value only is intended and the result has units of psi (MPa).

STACK BOND. The placement of masonry units in a bond pattern is such that head joints in successive courses are vertically aligned. For the purpose of this code, requirements for stack bond shall apply to masonry laid in other than running bond.

STIRRUP. Shear reinforcement in a beam or flexural member.

STONE MASONRY. Masonry composed of field, quarried or cast stone units bonded by mortar.

Ashlar stone masonry. Stone masonry composed of rectangular units having sawed, dressed, or squared bed surfaces and bonded by mortar.

Rubble stone masonry. Stone masonry composed of irregular-shaped units bonded by mortar.

STRENGTH

Design strength. Nominal strength multiplied by a strength reduction factor.

Nominal strength. Strength of a member or cross section calculated in accordance with these provisions before application of any strength reduction factors.

TIE, LATERAL. Loop of reinforcing bar or wire enclosing longitudinal reinforcement.

TIE, WALL. A connector that connects wythes of masonry walls together.

TILE. A ceramic surface unit, usually relatively thin in relation to facial area, made from clay or a mixture of clay or other ceramic materials, called the body of the tile, having either a "glazed" or "unglazed" face and fired above red heat in the course of manufacture to a temperature sufficiently high enough to produce specific physical properties and characteristics.

TILE, STRUCTURAL CLAY. A hollow masonry unit composed of burned clay, shale, fire clay or mixture thereof, and having parallel cells.

WALL. A vertical element with a horizontal length to thickness ratio greater than 3, used to enclose space.

Cavity wall. A wall built of masonry units or of concrete, or a combination of these materials, arranged to provide an air space within the wall, and in which the inner and outer parts of the wall are tied together with metal ties.

Composite wall. A wall built of a combination of two or more masonry units of different materials bonded together, one forming the backup and the other the facing elements.

Dry-stacked, surface-bonded wall. A wall built of concrete masonry units where the units are stacked dry, without mortar on the bed or head joints, and where both sides of the wall are coated with a surface-bonding mortar.

Masonry-bonded hollow wall. A wall built of masonry units so arranged as to provide an air space within the wall, and in which the facing and backing of the wall are bonded together with masonry units.

Parapet wall. The part of any wall entirely above the roof line.

WALL FRAME. A moment frame of masonry beams and masonry columns within a plane, with special reinforcement details and connections that provide resistance to lateral and gravity loads.

WEB. An interior solid portion of a hollow-masonry unit as placed in masonry.

WYTHE. Each continuous, vertical section of a wall, one masonry unit in thickness.

NOTATIONS

A_b = Cross-sectional area of anchor bolt, square inches (mm^2).

A_e = Effective area of masonry, square inches (mm^2).

A_g = Gross area of wall, square inches (mm^2).

A_{jh} = Total area of special horizontal reinforcement through wall frame joint, square inches (mm^2).

A_{mv} = Net area of masonry section bounded by wall thickness and length of section in direction of shear force considered, square inches (mm^2).

A_n = Net cross-sectional area of masonry, square inches (mm^2).

A_p = Projected area of masonry surface of a right circular cone for anchor bolt, square inches (mm^2).

A_s = Effective cross-sectional area of reinforcement, square inches (mm^2).

A_{se} = Effective area of reinforcement, square inches (mm^2).

A_{sh} = Total cross-sectional area of rectangular tie reinforcement for confined core, square inches (mm^2).

A_v = Cross-sectional area of shear reinforcement, square inches (mm^2).

a = Depth of equivalent rectangular stress block, inches (mm).

a_b = Depth of equivalent rectangular stress block at balanced strain conditions, inches (mm).

B_a = Design axial strength of an anchor bolt, pounds (N).

B_v = Design shear strength of an anchor bolt, pounds (N).

b = Effective width of rectangular member or width of flange for T and I sections, inches (mm).

b_a = Factored axial force on an anchor bolt, pounds (N).

b_v = Factored shear force on an anchor bolt, pounds (N).

b_w = Web width, inches (mm).

C_d = Deflection amplification factor as given in Table 1617.6.

c = Distance from neutral axis to the fiber of maximum compressive strain, inches (mm).

D = Dead loads, or related internal moments and forces.

d = Distance from compression face of flexural member to centroid of longitudinal tensile reinforcement, inches (mm).

d_b = Diameter of reinforcement, inches (mm).

d_{bb} = Diameter of largest beam longitudinal reinforcing bar passing through, or anchored in, a joint, inches (mm).

d_{bp} = Diameter of largest pier longitudinal reinforcing bar passing through a joint, inches (mm).

d_v = Length of member in direction of shear force, inches (mm).

E = Load effects of earthquake, or related internal moments and forces.

E_m = Modulus of elasticity of masonry, pounds per square inch (MPa).

E_s = Modulus of elasticity of steel, psi (GPa).

E_v = Modulus of rigidity of masonry, pounds per square inch (MPa).

e = Eccentricity of P_{uf}, inches (mm).

e_{mu} = Maximum usable compressive strain of masonry.

f_r = Modulus of rupture, pounds per square inch (MPa).

f_y = Specified yield stress of the reinforcement or the anchor bolt, pounds per square inch (MPa).

f_{yh} = Specified tensile yield stress of horizontal reinforcement, pounds per square inch (MPa).

f'_g = Specified compressive strength of grout at age of 28 days, pounds per square inch (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, pounds per square inch (MPa).

h = Effective height of a column, pilaster or wall, inches (mm).

h_b = Beam depth in the plane of the wall frame, inches (mm).

h_c = Cross-sectional dimension of grouted core measured center to center of confining reinforcement,inches (mm).

h_n = Height of structure above the base level to Level n, feet (m).

h_p = Pier depth in plane of wall frame, inches (mm).

I_{eff} = Effective moment of inertia, $inches^4$ (mm^4).

I_g, I_{cr} = Gross, cracked moment of inertia of wall cross section, $inches^4$ (mm^4).

I_n = Moment of inertia of the net cross-sectional area of a member, $inches^4$ (mm^4).

K = The lesser of the masonry cover, clear spacing between adjacent reinforcement, or 3 times d_b, inches (mm).

L = Live loads, or related internal moments and forces.

L_c = Length of coupling beam between coupled shear walls, inches (mm).

L_s = Distance between supports, inches (mm).

L_w = Length of wall, inches (mm).

l = Length of wall or segment, inches (mm).

l_b = Effective embedment depth of anchor bolt, inches (mm).

l_{be} = Anchor bolt edge distance, the least distance measured from edge of masonry to surface of anchor bolt,inches (mm).

l_d = Required development length of reinforcement, inches (mm).

l_{de} = Embedment length of reinforcement, inches (mm).

l_{dh} = Equivalent development length for a standard hook, inches (mm).

l_{ld} = Minimum lap splice length, inches (mm).

M = Moment on a masonry section due to unfactored load, inch-pounds (N-mm).

M_a = Maximum moment in member due to the applied loading for which defection is computed, inch-pounds (N-mm).

M_{cr} = Nominal cracking moment strength of masonry, inch-pounds (N-mm).

M_d = Design moment strength, inch-pounds (N-mm).

M_n = Nominal moment strength, inch-pounds (N-mm).

M_{ser} = Service moment at midheight of panel, including P effects, inch-pounds (N-mm).

M_u = Factored moment, inch-pounds (N-mm).

M_1, M_2 = Nominal moment strength at the ends of the cou pling beam, inch-pounds (N-mm).

N_v = Force acting normal to shear surface, pounds (N).

P = Axial force on a masonry section due to unfactored loads, pounds (N).

P_b = Nominal balanced design axial strength, pounds (N).

P_f = Load from tributary floor or roof area, pounds (N).

P_n = Nominal axial strength in masonry, pounds (N).

P_o = Nominal axial strength without bending, pounds (N).

P_u = Factored axial strength due to factored loads, pounds (N).

P_{uf} = Factored load from tributary floor or roof area, pounds (N).

P_{uw} = Factored weight of wall tributary to section under consideration, pounds (N).

P_w = Weight of wall tributary to section under consideration, pounds (N).

r = Radius of gyration, inches (mm).

S = Uncracked section modulus, inches^3 (mm^3).

s = Spacing of stirrups or of bent bars in direction parallel to that of main reinforcement, inches (mm).

t = Specified wall thickness dimension or at least lateral dimension of a column, inches (mm).

V = Shear on a masonry section due to unfactored loads, pounds (N).

V_g = Shear force due to gravity loads, pounds (N).

V_{jh} = Total horizontal joint shear, pounds (N).

V_m = Shear strength provided by masonry, pounds (N).

V_n = Nominal shear strength, pounds (N).

V_s = Shear strength provided by shear reinforcement, pounds (N).

V_u = Required shear strength due to factored loads, pounds (N).

W = Wind load, or related internal moments in forces.

w_u = Factored distributed lateral load.

γ = Reinforcement size factor.

Δ = Design story drift as determined in Section 1617.3, inches (mm).

Δ_a = Allowable story drift as specified in Table 1617.3, inches (mm).

Δ_s = Horizontal deflection at midheight under service load, inches (mm).

Δ_u = Deflection due to factored loads, inches (mm).

δ_{max} = The maximum displacement at Level x, inches (mm).

ρ = Ratio of area of flexural tensile reinforcement, A_s, to area bd.

ρ_b = Reinforcement ratio producing balanced strain conditions.

ρ_n = Ratio of distributed shear reenforcement on plane perpendicular to plane of A_{mv}.

ϕ = Strength reduction factor.

SECTION 2103 MASONRY CONSTRUCTION MATERIALS

2103.1 Concrete masonry units. Concrete masonry units shall conform to the following standards: ASTM C 55 for concrete brick; ASTM C 73 for calcium silicate face brick; ASTM C 90 for load-bearing concrete masonry units; or ASTM C 744 for prefaced concrete and calcium silicate masonry units.

2103.2 Clay or shale masonry units. Clay or shale masonry units shall conform to the following standards: ASTM C 34 for structural clay load-bearing wall tile; ASTM C 56 for structural clay nonload-bearing wall tile; ASTM C 62 for building brick (solid masonry units made from clay or shale); ASTM C 1088 for solid units of thin veneer brick; ASTM C 126 for ceramic-glazed structural clay facing tile, facing brick and solid masonry units; ASTM C 212 for structural clay facing tile; ASTM C 216 for facing brick (solid masonry units made from clay or shale); and ASTM C 652 for hollow brick (hollow masonry units made from clay or shale).

Exception: Structural clay tile for nonstructural use in fireproofing of structural members and in wall furring shall not be required to meet the compressive strength specifications.

The fire-resistance rating shall be determined in accordance with ASTM E 119 and shall comply with the requirements of Table 602.

2103.3 Stone masonry units. Stone masonry units shall conform to the following standards: ASTM C 503 for marble building stone (exterior): ASTM C 568 for limestone building stone; ASTM C 615 for granite building stone; ASTM C 616 for sandstone building stone; or ASTM C 629 for slate building stone.

Comm 62.2103 Cast-stone masonry units.

(1) Cast-stone masonry units covered under this category are homogeneous or faced, dry cast concrete products other than conventional concrete masonry units (brick or block), but of similar size.

(2) Cast-stone masonry units shall be made with portland cement, water and suitable mineral aggregates, with or without admixtures, and reinforced if required.

(3) Cast-stone masonry units shall have a minimum compressive strength of 6500 psi and a maximum water absorption of 6 percent when tested as 2-inch by 2-inch (51 mm × 51 mm) cylinders or cubes.

2103.4 Ceramic tile. Ceramic tile shall be as defined in ANSI A137.1 and shall conform to the requirements of ANSI A137.1.

2103.5 Glass unit masonry. Hollow glass units shall be partially evacuated and have a minimum average glass face thickness of $^3/_{16}$ inch (4.8 mm). Solid glass block units shall be provided when required. The surfaces of units intended to be in contact with mortar shall be treated with a polyvinyl butyral coating or latex-based paint. Reclaimed units shall not be used.

2103.6 Second-hand units. Second-hand masonry units shall not be reused unless the units conform to the requirements of new units. The units shall be of whole, sound materials and be free from cracks and other defects that will interfere with proper laying or use. Old mortar shall be cleaned from the unit before reuse.

2103.7 Mortar. Mortar for use in masonry construction shall conform to ASTM C 270 and shall conform to the proportion specifications of Table 2103.7(1) or the property specifications of Table 2103.7(2). Type S or N mortar shall be used for glass unit masonry. The amount of water used in mortar for glass unit masonry shall be adjusted to account for the lack of absorption. Retempering of mortar for glass unit masonry shall not be permitted after initial set. Unused mortar shall be discarded within $2^1/_2$ hours after initial mixing except that unused mortar for glass unit masonry shall be discarded within $1^1/_2$ hours after initial mixing.

2103.8 Surface-bonding mortar. Surface-bonding mortar shall comply with ASTM C 887. Surface bonding of concrete masonry units shall comply with ASTM C 946.

2103.9 Mortars for ceramic wall and floor tile. Portland cement mortars for installing ceramic wall and floor tile shall comply with ANSI A108.1A and A108.1B and be of the compositions indicated in Table 2103.9.

TABLE 2103.7(1)
MORTAR PROPORTIONS

MORTAR	TYPE	PROPORTIONS BY VOLUME (cementitious materials)							HYDRATED LIME OR LIME PUTTY	AGGREGATE MEASURED IN A DAMP, LOOSE CONDITION
		Portland cement[a] or blended cement[b]	Masonry cement[c]			Mortar cement[d]				
			M	S	N	M	S	N		
Cement-lime	M	1	—	—	—	—	—	—	$^1/_4$	Not less than $2^1/_4$ and not more than 3 times the sum of the separate volumes of cementitious materials
	S	1	—	—	—	—	—	—	over $^1/_4$ to $^1/_2$	
	N	1	—	—	—	—	—	—	over $^1/_2$ to $1^1/_4$	
	O	1	—	—	—	—	—	—	over $1^1/_4$ to $2^1/_2$	
Mortar cement	M	1	—	—	—	—	—	1	—	
	M	—	—	—	—	1	—	—	—	
	S	$^1/_2$	—	—	—	—	—	1	—	
	S	—	—	—	—	—	1	—	—	
	N	—	—	—	—	—	—	1	—	
	O	—	—	—	—	—	—	1	—	
Masonry cement	M	1	—	—	—	—	—	—	—	
	M	—	1	—	—	—	—	—	—	
	S	$^1/_2$	—	—	—	—	—	—	—	
	S	—	—	1	—	—	—	—	—	
	N	—	—	—	—	—	—	—	—	
	O	—	—	—	—	—	—	—	—	

a. Portland cement conforming to the requirements of ASTM C 150.
b. Blended cement conforming to the requirements of ASTM C 595.
c. Masonry cement conforming to the requirements of ASTM C 91.
d. Mortar cement conforming to the requirements of ASTM C 1329.

**TABLE 2103.7(2)
MORTAR PROPERTIES[a]**

MORTAR	TYPE	AVERAGE COMPRESSIVE[b] STRENGTH AT 28 DAYS minimum (psi)	WATER RETENTION minimum (%)	AIR CONTENT maximum (%)
Cement-lime	M	2,500	75	12
	S	1,800	75	12
	N	750	75	14[c]
	O	350	75	14[c]
Mortar cement	M	2,500	75	12
	S	1,800	75	12
	N	750	75	14[c]
	O	350	75	14[c]
Masonry cement	M	2,500	75	18
	S	1,800	75	18
	N	750	75	20[d]
	O	350	75	20[d]

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 6.895 kPa.

a. This aggregate ratio (measured in damp, loose condition) shall not be less than $2^1/_4$ and not more than 3 times the sum of the separate volumes of cementitious materials.

b. Average of three 2-inch cubes of laboratory prepared mortar, in accordance with ASTM C 270.

c. When structural reinforcement is incorporated incement-lime or mortar cement mortars, the maximum air content shall not exceed 12 percent.

d. When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall not exceed 18 percent.

**TABLE 2103.9
CERAMIC TILE MORTAR COMPOSITIONS**

LOCATION	MORTAR	COMPOSITION
Walls	Scratchcoat	1 cement; $^1/_5$ hydrated lime; 4 dray or 5 damp sand
	Setting bed and leveling coat	1 cement; $^1/_2$ hydrated lime; 5 damp sand to 1 cement 1 hydrated lime, 7 damp sand
Floors	Setting bed	1 cement; $^1/_{10}$ hydrated lime; 5 dry or 6 damp sand; or 1 cement; 5 dry or 6 damp sand
Ceilings	Scratchcoat and sand bed	1 cement; $^1/_2$ hydrated lime; $2^1/_2$ dry sand or 3 damp sand

2103.9.1 Dry-set portland cement mortars. Premixed prepared portland cement mortars, which require only the addition of water and which are used in the installation of ceramic tile, shall comply with ANSI A118.1. The shear bond strength for tile set in such mortar shall be as required in accordance with ANSI A118.1. Tile set in dry-set portland cement mortar shall be installed in accordance with ANSI A108.5.

2103.9.2 Electrically conductive dry-set mortars. Premixed prepared portland cement mortars, which require only the addition of water and which comply with ANSI A118.2, shall be used in the installation of electrically conductive ceramic tile. Tile set in electrically conductive dry-set mortar shall be installed in accordance with ANSI A108.7.

2103.9.3 Latex-modified portland cement mortar. Latex-modified portland cement thin-set mortars in which latex is added to dry-set mortar as a replacement for all or part of the gauging water that are used for the installation of ceramic tile shall comply with ANSI A118.4. Tile set in latex-modified portland cement shall be installed in accordance with ANSI A108.5.

2103.9.4 Epoxy mortar. Ceramic tile set and grouted with chemical-resistant epoxy shall comply with ANSI A118.3. Tile set and grouted with epoxy shall be installed in accordance with ANSI A108.6.

2103.9.5 Furan mortar and grout. Chemical-resistant furan mortar and grout that are used to install ceramic tile shall comply with ANSI A118.5. Tile set and grouted with furan shall be installed in accordance with ANSI A108.8.

2103.9.6 Modified epoxy-emulsion mortar and grout. Modified epoxy-emulsion mortar and grout that are used to install ceramic tile shall comply with ANSI A118.8. Tile set and grouted with modified epoxy-emulsion mortar and grout shall be installed in accordance with ANSI A108.9.

2103.9.7 Organic adhesives. Water-resistant organic adhesives used for the installation of ceramic tile shall comply with ANSI A136.1. The shear bond strength after water immersion shall not be less than 40 psi (275 kPa) for Type I adhesive, and not less than 20 psi (138 kPa) for Type II adhesive, when tested in accordance with ANSI A136.1. Tile set in organic adhesives shall be installed in accordance with ANSI A108.4.

2103.9.8 Portland cement grouts. Portland cement grouts used for the installation of ceramic tile shall comply with ANSI A118.6. Portland cement grouts for tile work shall be installed in accordance with ANSI A108.10.

2103.10 Grout. Grout shall conform to Table 2103.10 or to ASTM C 476. When grout conforms to ASTM C 476, the grout shall be specified by proportion requirements or property requirements.

TABLE 2103.10
GROUT PROPORTIONS BY VOLUME FOR MASONRY CONSTRUCTION

TYPE	PARTS BY VOLUME OF PORTLAND CEMENT OR BLENDED CEMENT	PARTS BY VOLUME OF HYDRATED LIME OR LIME PUTTY	AGGREGATE, MEASURED IN A DAMP, LOOSE CONDITION	
			Fine	Coarse
Fine grout	1	0-$^1/_{10}$	2$^1/_4$-3 times the sum of the volumes of the cementitious materials	—
Coarse grout	1	0-$^1/_{10}$	2$^1/_4$-3 times the sum of the volumes of the cementitious materials	1-2 times the sum of the volumes of the cementitious materials

2103.11 Metal reinforcement and accessories. Metal reinforcement and accessories shall conform to Sections 2103.11.1 through 2103.11.7.

2103.11.1 Deformed reinforcing bars. Deformed reinforcing bars shall conform to the following standards: ASTM A 615 for deformed and plain billet-steel bars for concrete reinforcement; ASTM A 616 for rail-steel deformed and plain bars for concrete reinforcement; ASTM A 617 for axle-steel deformed and plain bars for concrete reinforcement; ASTM A 706 for low-alloy-steel deformed bars for concrete reinforcement; ASTM A 767 for zinc-coated reinforcing steel bars; and ASTM A 775 for epoxy-coated reinforcing steel bars.

2103.11.2 Joint reinforcement. Joint reinforcement shall comply with ASTM A 951.

2103.11.3 Deformed reinforcing wire. Deformed reinforcing wire shall conform to ASTM A 496.

2103.11.4 Wire fabric. Wire fabric shall conform to ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement or ASTM A 496 for welded deformed steel wire fabric for concrete reinforcement.

2103.11.5 Anchors, ties and accessories. Anchors, ties and accessories shall conform to the following standards: ASTM A 36 for structural steel; ASTM A 82 for plain steel wire for concrete reinforcement; ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement; ASTM A 167, Type 304, for stainless and heat-resisting chromium-nickel steel plate, sheet and strip; and ASTM A 366 for cold-rolled carbon steel sheet, commercial quality.

2103.11.6 Corrosion protection. Joint reinforcement shall be protected from corrosion by galvanizing in accordance with ASTM A 951. Anchors, wall ties and accessories, except those of Type 304 stainless steel complying with ASTM A 167, shall be protected from corrosion by galvanizing as follows.

Metal accessories for use in exterior wall construction or interior walls exposed to a mean relative humidity exceeding 75 percent shall be hot-dipped galvanized after fabrication with a minimum coating of 1.5 ounces per square foot (458 g/m^2) in accordance with ASTM A 153. Metal accessories for use in interior wall construction shall be mill galvanized with a minimum coating of 0.1 ounce per square foot (31 g/m^2) in accordance with ASTM A 641 for wire anchors and ties; and Class G-60 for sheet metal anchors and ties.

2103.11.7 Tests. Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.

SECTION 2104 CONSTRUCTION

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 through 2104.5 and with ACI 530.1/ASCE 6/TMS 602.

2104.1.1 Tolerances. Masonry, except masonry veneer, shall be constructed within the tolerances specified in ACI 530.1/ASCE 6/TMS 602.

2104.1.2 Placing mortar and units. Placement of mortar and units shall comply with Sections 2104.1.2.1 through 2104.1.2.5.

2104.1.2.1 Bed and head joints. Unless otherwise required or indicated on the construction documents, head and bed joint shall be $^3/_8$ inch (9.5 mm) thick, except that the thickness of the bed joint of the starting course placed over foundations shall not be less than $^1/_4$ inch (6.4 mm) and not more than $^3/_4$ inch (19.1 mm).

2104.1.2.1.1 Open-end units. Open-end units with beveled ends shall be fully grouted. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key that permits grouts within $^5/_8$ inch (15.9 mm) of the face of the unit. The units shall be tightly butted to prevent leakage of the grout.

2104.1.2.2 Hollow units. Hollow units shall be placed such that face shells of bed joints are fully mortared; webs are fully mortared in all courses of piers, columns, pilasters, in the starting course on foundations where adjacent cells or cavities are to be grouted, and where otherwise required; and head joints are mortared a minimum distance from each face equal to the face shell thickness of the unit.

2104.1.2.3 Solid units. Unless otherwise required or indicated on the construction documents, solid units shall be placed in fully mortared bed and head joints. The ends of the units shall be completely buttered. Head joints shall not be filled by slushing with mortar. Head joints shall be constructed by shoving mortar tight against the adjoining unit. Bed joints shall not be furrowed deep enough to produce voids.

2104.1.2.4 Glass unit masonry. Glass units shall be placed so head and bed joints are filled solidly. Mortar shall not be furrowed.

Unless otherwise required, head and bed joints of glass unit masonry shall be $^1/_4$ inch (6.4 mm) thick, except that vertical joint thickness of radial panels shall not be less than $^1/_8$ inch (3.2 mm). The bed joint thickness tolerance shall be minus $^{1}/_{16}$ inch (1.6 mm) and plus $^{1}/_{8}$ inch (3.2 mm). The head joint thickness tolerance shall be plus or minus $^{1}/_{8}$ inch (3.2 mm).

2104.1.2.5 All units. Units shall be placed while the mortar is soft and plastic. Any unit disturbed to the extent that the initial bond is broken after initial positioning shall be removed and relaid in fresh mortar.

2104.1.3 Installation of wall ties. The ends of wall ties shall be embedded in mortar joints. Wall tie ends shall engage outer face shells of hollow units by at least $^{1}/_{2}$ inch (12.7 mm). Wire wall ties shall be embedded at least $1^{1}/_{2}$ inch (38 mm) into the mortar bed of solid masonry units or solid-grouted hollow units. Wall ties shall not be bent after being embedded in grout or mortar.

2104.1.4 Chases and recesses. Chases and recesses shall be constructed as masonry units are laid. Masonry directly above chases or recesses wider than 12 inches (305 mm) shall be supported on lintels.

2104.1.5 Lintels. The design for lintels shall be in accordance with the masonry design provisions of either Section 2107 or 2108. Minimum length of end support shall be 4 inches (102 mm).

2104.1.6 Support on wood. Masonry shall not be supported on wood girders or other forms of wood construction except as permitted in Section 2304.12.

2104.1.7 Masonry protection. The top of unfinished masonry work shall be covered to protect the masonry from the weather.

2104.1.8 Weep holes. Weep holes provided in the outside wythe of masonry walls shall be at a maximum spacing of 33 inches (838 mm) on center. Weep holes shall not be less than $^{3}/_{16}$ inch (4.8 mm) in diameter.

2104.2 Corbeled masonry. The maximum corbeled projection beyond the face of the wall shall not be more than one-half of the wall thickness nor one-half the wythe thickness for hollow walls. The maximum projection of one unit shall neither exceed one-half the height of the unit nor one-third the thickness at right angles to the wall.

2104.2.1 Molded cornices. Unless structural support and anchorage are provided to resist the overturning moment, the center of gravity of projecting masonry or molded cornices shall lie within the middle one-third of the supporting wall. Terra cotta and metal cornices shall be provided with a structural frame of approved noncombustible material anchored in an approved manner.

2104.3 Cold-weather construction. The following cold-weather procedures shall be implemented when either the ambient temperature falls below 40°F (4°C) or the temperature of masonry units is below 40°F (4°C):

1. Temperatures of masonry units shall not be less than 20°F (-7°C) when laid in the masonry. Visible ice on masonry units shall be removed before the unit is laid in the masonry.
2. Mortar sand or mixing water shall be heated to produce mortar temperatures between 40°F (4°C) and 120°F (49°C) at the time of mixing. Mortar shall be maintained above freezing until used in masonry.
3. Heat sources shall be used where ambient temperatures are between 20°F (-7°C) and 25°F (-4°C) on both sides of the masonry under construction and wind breaks shall be installed when wind velocity is in excess of 15 mph (24 km/hr).
4. Where ambient temperatures are below 20°F (-7°C), an enclosure for the masonry under construction shall be provided and heat sources shall be used to maintain temperatures above 32°F (0°C) within the enclosure.
5. Where mean daily temperatures are between 32°F (0°C) and 40°F (4°C), completed masonry shall be protected from rain or snow by covering with a weather-resistant membrane for 24 hours after construction.
6. Where mean daily temperatures are between 25°F (-4°C) and 32°F (0°C), completed masonry shall be completely covered with a weather-resistant membrane for 24 hours after construction.
7. Where mean daily temperatures are between 20°F(-7°C) and 25°F (-4°C), completed masonry shall be completely covered with insulating blankets or equal protection for 24 hours after construction.
8. Where mean daily temperatures are below 20°F (-7°C), masonry temperature shall be maintained above 32°F (0°C) for 24 hours after construction by enclosure with supplementary heat, by electric heating blankets, by infrared heat lamps or by other approved methods.
9. Glass unit masonry shall not be laid during cold periods as defined in this section. The temperature of glass unit masonry shall be maintained above 40°F (4°C) for the first 48 hours after construction.

2104.4 Hot weather construction. The following hot-weather procedures shall be implemented when the temperature or the temperature and wind-velocity limits of this section are exceeded.

2104.4.1 Preparation. The following requirements shall be met prior to conducting masonry work.

2104.4.1.1 Temperature. When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (13 km/h):

1. Necessary conditions and equipment shall be provided to produce mortar having a temperature below 120°F (49°C).
2. Sand piles shall be maintained in a damp, loose condition.

2104.4.1.2 Special conditions. When the ambient temperature exceeds 115°F (46°C), or 105°F (40°C) with a wind velocity greater than 8 mph (13 km/h), the requirements of Section 2104.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.

2104.4.2 Construction. The following requirements shall be met while masonry work is in progress.

2104.4.2.1 Temperature. When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (13 km/h):

1. The temperature of mortar and grout shall be maintained below 120°F (49°C).
2. Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.
3. Mortar consistency shall be maintained by retempering with cool water.
4. Mortar shall be used within 2 hours of initial mixing.

2104.4.2.2 Special conditions. When the ambient temperature exceeds 115°F (46°C), or exceeds 105°F (40°C) with a wind velocity greater than 8 mph (13 km/h), the requirements of Section 2104.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.

2104.4.3 Protection. When the mean daily temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (13 km/h), newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.

2104.5 Wetting of brick. Brick (clay or shale) at the time of laying shall require wetting if the unit's initial rate of water absorption exceeds 30 grams per 30 square inches (19 355 mm^2) per minute or 0.035 ounce per square inch (1 g/645 mm^2), as determined by ASTM C 67.

SECTION 2105
QUALITY ASSURANCE

2105.1 Deleted.

2105.2 Acceptance relative to strength requirements.

2105.2.1 Compliance with f'_m. Compressive strength of masonry shall be considered satisfactory if the compressive strength of each masonry wythe and grouted collar joint equals or exceeds the value of f'_m.

2105.2.2 Determination of compressive strength. The compressive strength for each wythe shall be determined by the unit strength method or by the prism test method as specified herein.

2105.2.2.1 Unit strength method.

2105.2.2.1.1 Clay masonry. The compressive strength of masonry shall be determined based on the strength of the units and the type of mortar specified using Table 2105.2.2.1.1, provided:

1. Units conform to ASTM C 62, ASTM C 216 or ASTM C 652 and are sampled and tested in accordance with ASTM C 67.
2. Thickness of bed joints does not exceed $^5/_8$ inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   - 3.1. Grout conforms to ASTM C 476.
   - 3.2. Minimum grout compressive strength equals f'_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

TABLE 2105.2.2.1.1
COMPRESSIVE STRENGTH OF CLAY MASONRY

NET AREA COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS (psi)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (psi)
Type M or S mortar	Type N mortar	
2,400	3,000	1,000
4,400	5,500	1,500
6,400	8,000	2,000
8,400	10,500	2,500
10,400	13,000	3,000
12,400	—	3,500
14,400	—	4,000

For SI: 1 pound per square inch = 0.00689 MPa.

2105.2.2.1.2 Concrete masonry. The compressive strength of masonry shall be determined based on the strength of the unit and type of mortar specified using Table 2105.2.2.1.2, provided:

1. Units conform to ASTM C 55 or ASTM C 90 and are sampled and tested in accordance with ASTM C 140.
2. Thickness of bed joints does not exceed $^5/_8$ inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   - 3.1. Grout conforms to ASTM C 476.
   - 3.2. Minimum grout compressive strength equals f'_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

TABLE 2105.2.2.1.2
COMPRESSIVE STRENGTH OF CONCRETE MASONRY

NET AREA COMPRESSIVE STRENGTH OF CONCRETE MASONRY UNITS (psi)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (psi)[a]
Type M or S mortar	Type N mortar	
1,250	1,300	1,000
1,900	2,150	1,500
2,800	3,050	2,000
3,750	4,050	2,500
4,800	5,250	3,000

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 0.00689 MPa.
a. For units of less than 4 inches in height, 85 percent of the values listed.

2105.2.2.2 Prism test method.

2105.2.2.2.1 General. The compressive strength of masonry shall be determined by the prism test method:

1. Where specified in the construction documents.
2. Where masonry does not meet the requirements for application of the unit strength method in Section 2105.2.2.1.

2105.2.2.2.2 Number of prisms per test. A prism test shall consist of testing three prisms in accordance with ASTM C 1314.

2105.2.2.2.3 Compressive strength determination. The compressive strength of masonry shall be taken as the average strength of three prisms, as modified in ASTM C 1314, but not more than the strength of the masonry units used in prism construction.

2105.3 Testing prisms from constructed masonry. When approved by the building official, acceptance of masonry that does not meet the requirements of Section 2105.2.2.1 or 2105.2.2.2 shall be permitted to be based on tests of prisms cut from the masonry construction in accordance with Sections 2105.3.1, 2105.3.2 and 2105.3.3.

2105.3.1 Prism sampling and removal. A set of three masonry prisms that are at least 28 days old shall be saw cut from the masonry for each 5,000 square feet (465 m^2) of the wall area that is in question but not less than one set of three masonry prisms for the project. The length, width and height dimensions of the prisms shall comply with the requirements of ASTM C 1314. Transporting, preparation and testing of prisms shall be in accordance with ASTM C 1314.

2105.3.2 Compressive strength calculations. The compressive strength of prisms shall be the value calculated in accordance ASTM C 1314, except that the net cross-sectional area of the prism shall be based on the net mortar bedded area.

2105.3.3 Compliance. Compliance with the requirement for the specified compressive strength of masonry, f'_m, shall be considered satisfied provided the modified compressive strength equals or exceeds the specified f'_m. Additional testing of specimens cut from locations in question shall be permitted.

2105.4 Mortar testing. When required, mortar shall be tested in accordance with the property specifications of ASTM C 270 or evaluated in accordance with ASTM C 780.

2105.5 Grout testing. When required, grout shall be tested in accordance with ASTM C 1019.

SECTION 2106
SEISMIC DESIGN

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section 2106.1.1, 2106.2, 2106.3, 2106.4, 2106.5 or 2106.6 depending on the structure's seismic design category as defined in Section 1613.3, except that masonry structures designed by the working stress design method shall be permitted to comply with Section 2106.1.2.

2106.1.1 Basic seismic-force-resisting system. Buildings relying on masonry shear walls or on masonry wall frames as part of the basic seismic-force-resisting system shall comply with Section 2106.1.1.1, 2106.1.1.2, 2106.1.1.3, 2106.1.1.4, 2106.1.1.5 or 2106.1.1.6.

Exception: Buildings assigned to Seismic Design Category A are permitted to have shear walls complying with Section 5.3 ACI 530/ASCE 5/TMS 402 or with Section 2109.2.1.

Shear walls having a response modification factor, *R*, as defined in Table 1616.1, of at least that of the shear wall types required in Sections 2106.3.1 and 2106.4.2.3 are permitted.

2106.1.1.1 Ordinary plain masonry shear walls. Ordinary plain masonry shear walls shall comply with the requirements of Section 2.2 of ACI 530/ASCE 5/TMS 402 or Section 2108.10.

2106.1.1.2 Ordinary reinforced masonry shear walls. Ordinary reinforced masonry shear walls shall comply with the requirements of Section 2106.4.2.3.1 and shall be designed by Section 2.3 of ACI 530/ASCE 5/TMS 402 or Section 2108.9.

2106.1.1.3 Detailed plain masonry shear walls. Detailed plain masonry shear walls shall comply with the requirements of Section 2106.4.2.3.1 and shall be designed by Section 2.2 of ACI 530/ASCE 5/TMS 402 or by Section 2108.10.

2106.1.1.4 Intermediate reinforced masonry shear walls. Intermediate reinforced masonry shear walls shall comply with the requirements of Section 2106.4.2.3.1 and shall be designed by Section 2.3 of ACI 530/ASCE 5/TMS 402 or Section 2108.9. In addition, the maximum spacing of vertical reinforcement shall not exceed 48 inches (1219 mm).

2106.1.1.5 Special reinforced masonry shear walls. Special reinforced masonry shear walls shall comply with the requirements of Section 2106.5.3.1 and shall be designed by Section 2.3 of ACI 530/ASCE 5/TMS 402 or Section 2108.9.

2106.1.1.6 Masonry wall frames. Masonry wall frames shall comply with Section 2108.9.6.

2106.1.2 Alternate seismic design requirements for structures designed by the working stress design method. Masonry members and structures designed by the working stress design provisions of Section 2107 shall be permitted to comply with the seismic design and construction requirements of Section 1.13 of the ACI 530/ASCE 5/TMS 402 except as modified in Sections 2106.1.2.1, 2106.1.2.2, 2106.1.2.3 and 2106.1.2.4.

2106.1.2.1 ACI 530/ASCE 5/TMS 402, Section 2.1.1.2.3. Section 2.1.1.2.3 of ACI 530/ASCE 5/TMS 402 shall not apply to the design of masonry structures.

2106.1.2.2 ACI 530/ASCE 5/TMS 402, Sections 1.13.2, 1.13.3, 1.13.3.1, 1.13.4, 1.13.4.1, 1.13.5, 1.13.5.1, 1.13.6, 1.13.6.1, 1.13.7 and 1.13.7.1. Requirements for Seismic Performance Category A, B, C or D as described in Section 1.13 of ACI 530/ASCE 5/TMS 402, shall apply to structures in Seismic Design Category A, B, C or D, as defined in Section 1616, respectively. Requirements for Seismic Performance Category E as described in Section 1.13 of ACI 530/ASCE 5/TMS 402 shall apply to structures in Seismic Design Category E or F as described in Section 1616.

2106.1.2.3 Response modification coefficients. The response modification coefficients, *R*, of Table 1617.6 for special reinforced masonry shear walls shall apply, provided masonry is designed in accordance with Section 2.3 and Section 1.13.6 of ACI 530/ASCE 5/TMS 402. The *R* coefficients of Table 1617.6 for intermediate reinforced masonry shear walls shall apply for masonry designed in accordance with Section 2.3 and Section 1.13.5 of ACI 530/ASCE 5/TMS 402. The *R* coefficients of Table 1617.6 for ordinary reinforced masonry shear walls shall apply for masonry designed in accordance with Section 2.3 of ACI 530/ASCE 5/TMS 402. The *R* coefficients of Table 1617.6 for detailed plain masonry shear walls shall apply for masonry designed in accordance with Section 2.2 and Section 1.13.5 of ACI 530/ASCE 5/TMS 402. The *R* coefficients of Table 1616 for ordinary plain masonry shear walls shall apply for all other masonry.

2106.1.2.4 Design loads for shear walls. When calculating shear or diagonal tension stresses, shear walls that resist seismic forces in Seismic Design Category D, E or F shall be designed to resist 1.5 times the forces required by Chapter 16.

2106.2 Seismic Design Category A. Structures assigned to Seismic Design Category A shall comply with the requirements of Section 2109 (empirical masonry design), 2108.10 (plain masonry design, strength method) or 2108.9 (reinforced masonry design, strength method), or with the requirements of ACI 530/ASCE 5/TMS 402, Chapter 5 (empirical masonry design), Section 2.2 (plain masonry design, working stress method) or Section 2.3 (reinforced masonry design, working stress method).

2106.2.1 Anchorage of masonry walls. Masonry walls shall be anchored to the roof and floors that provide lateral support for the wall in accordance with Section 1616.4.3.

2106.3 Seismic Design Category B. Structures assigned to Seismic Design Category B shall conform to the requirements for Seismic Design Category A.

2106.3.1 Masonry shear walls. Masonry shear walls shall comply with the requirements of ordinary plain masonry shear walls or ordinary reinforced masonry shear walls.

2106.4 Seismic Design Category C. Structures assigned to Seismic Design Category C shall conform to the requirements for Seismic Design Category B and to the additional requirements of this section.

2106.4.1 Design of elements that are not part of the lateral-force-resisting system.

2106.4.1.1 Load-bearing frames or columns. Load-bearing frames or columns that are not part of the lateral-force-resisting system shall be analyzed as to their effect on the response of the system. Such frames or columns shall be adequate for vertical-load-carrying capacity and induced moment due to the design story drift.

2106.4.1.2 Masonry partition walls. Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.

2106.4.1.3 Reinforcement requirements for masonry elements. Masonry elements listed in Section 2106.4.1.2 shall be reinforced in either the horizontal or vertical direction dependent upon the location of the lateral supporting elements in accordance with the following:

1. Horizontal joint reinforcement shall consist of at least two longitudinal W1.7 wires spaced not more than 16 inches (406 mm) for walls greater than 4 inches (102 mm) in width and at least one longitudinal W1.7 wire spaced not more 16 inches (406 mm) for walls not exceeding 4 inches (102 mm) in width; or at least one No. 4 bar spaced not more than 48 inches (1219 mm). Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest that the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 inches (406 mm) of the top and bottom of these masonry elements.
2. Vertical reinforcement shall consist of at least one No. 4 bar spaced not more than 48 inches (1219 mm). Vertical reinforcement shall be located within 16 inches (406 mm) of the ends of masonry walls.

2106.4.2 Design of elements that are part of the lateral-force-resisting system.

2106.4.2.1 Connections to masonry shear walls. Connectors shall be provided to transfer forces between masonry walls and horizontal elements in accordance with the requirements of ACI 530/ASCE 5/TMS 402, Section 2.1.6. Connectors shall be designed to transfer horizontal design forces acting either perpendicular or parallel to the wall, but not less than 200 pounds per lineal foot (2919 N/m) of wall. The maximum spacing between connectors shall be 4 feet (1219 mm).

2106.4.2.2 Connections to masonry columns. Connectors shall be provided to transfer forces between masonry columns and horizontal elements in accordance with the requirements of ACI 530/ASCE 5/TMS 402, Section 2.1.6. Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two No. 4 lateral ties provided in the top 5 inches (127 mm) of the column.

2106.4.2.3 Masonry shear walls. Masonry shear walls shall comply with the requirements for ordinary reinforced masonry shear walls or intermediate reinforced masonry shear walls.

2106.4.2.3.1 Minimum reinforcement requirements for masonry shear walls. Vertical reinforcement of at least 0.20 square inch (129 mm^2) in cross-sectional area shall be provided at corners, within 16 inches (406 mm) of each side of openings, within 8 inches (203 mm) of each side of movement joints, within 8 inches (203 mm) of the ends of walls, and at a maximum spacing of 10 feet (3048 mm).

Horizontal joint reinforcement shall consist of at least two wires of W1.7 spaced not more than 16 inches (406 mm); or bond beam reinforcement shall be provided of at least 0.2 square inch (129 mm^2) in cross-sectional area spaced not more than 10 feet (3048 mm). Horizontal reinforcement shall also be provided at the bottom and top of wall openings and shall extend not less than 24 inches (610 mm) nor less than 40 bar diameters past the opening; continuously at structurally connected roof and floor levels; and within 16 inches (406 mm) of the top of walls.

2106.4.2.4 Discontinuous members. Columns and pilasters supporting reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.

2106.5 Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to all of the requirements for Seismic Design Category C and the additional requirements of this section.

2106.5.1 Design requirements. Masonry elements other than those covered by Section 2106.4.1 shall be designed in accordance with the requirements of Section 2108.9 and Sections 2106.5.2 through 2106.5.6 or in accordance with Sections 2106.5.2 through 2106.5.6 and ACI 530/ASCE 5/TMS 402, Section 2.3.

2106.5.2 Minimum reinforcement requirements for masonry walls. Masonry walls other than those covered by Section 2106.4.1 shall be reinforced in both the vertical and horizontal direction. The sum of the cross-sectional area of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum cross-sectional area in each direction shall be not less than 0.0007 times the gross cross-sectional area of the wall. Reinforcement shall be uniformly distributed. The maximum spacing of reinforcement shall be 48 inches (1219 mm) except for stack bond masonry. Wythes of stack bond masonry shall be constructed of fully grouted hollow open-end units, fully grouted hollow units laid with full head joints or solid units. Maximum spacing of reinforcement for walls with stack bond masonry shall be 24 inches (610 mm).

2106.5.3 Masonry shear walls. Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

2106.5.3.1 Shear wall reinforcement requirements. Shear walls shall be reinforced in accordance with Section 2106.5.2 and the maximum spacing of vertical and horizontal reinforcement shall be the smaller of one-third the length of the shear wall, one-third the height of the shear wall, or 48 inches (1219 mm). The minimum cross-sectional area of vertical reinforcement shall be one-third of the required shear reinforcement. Shear reinforcement shall be anchored around vertical-reinforcing bars with a standard hook.

2106.5.3.2 Shear wall shear strength. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within a region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.

$$V_n = A_n \rho_n f_y \qquad \textbf{(Equation 21-1)}$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one-half story height.

For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.9.3.5.2.1.

2106.5.4 Minimum reinforcement for masonry columns. Lateral ties in masonry columns shall be spaced not more than 8 inches (203 mm) on center and shall be at least $^3/_8$ inch (9.5 mm) diameter. Lateral ties shall be embedded in grout.

2106.5.5 Material requirements. Neither Type N mortar nor masonry cement shall be used as part of the lateral-force-resisting system.

2106.5.6 Lateral tie anchorage. Standard hooks for lateral tie anchorage shall be either a 135-degree (2.36 rad) standard hook or a 180-degree (3.14 rad) standard hook.

2106.6 Seismic Design Category E or F. Structures assigned to Seismic Design Category E or F shall conform to the requirements of Seismic Design Category D and to the additional requirements and limitations of this section.

2106.6.1 Design of elements that are not part of the lateral-force-resisting system. Stack bond masonry that is not part of the lateral-force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0015 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 24 inches (610 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

2106.6.2 Design of elements that are part of the lateral-force-resisting system. Stack bond masonry that is part of the lateral-force-resisting system shall have a horizontal cross-sectional area of reinforcement of at least 0.0025 times the gross cross-sectional area of masonry. The maximum spacing of horizontal reinforcement shall be 16 inches (406 mm). These elements shall be solidly grouted and shall be constructed of hollow open-end units or two wythes of solid units.

2106.6.2.1 Masonry shear walls. Masonry shear walls shall comply with the requirements for special reinforced masonry shear walls.

2106.6.2.2 Reinforced hollow unit masonry. Reinforced hollow unit masonry shall conform to the following requirement. Reinforcement shall be secured against displacement prior to grouting at intervals not exceeding 112 bar diameters by wire positioners or other suitable devices.

SECTION 2107
WORKING STRESS DESIGN

2107.1 General. The design of masonry structures using working stress design shall comply with Section 2106 and the requirements of Chapters 1 and 2, except Section 2.1.1.1.1, of ACI 530/ASCE 5/TMS 402. The text of ACI 530/ASCE 5/TMS 402 shall be modified as follows.

2107.2 Modifications to ACI 530/ASCE 5/TMS 402.

2107.2.1 ACI 530/ASCE 5/TMS 402, Chapter 2. Special inspection during construction shall be provided as set forth in Section 1704.5.

2107.2.2 ACI 530/ASCE 5/TMS 402, Section 2.1.4. Masonry columns used only to support roofs of carports, porches, sheds or similar light structures assigned to Seismic Design Category A, B or C are permitted to be constructed as follows:

1. Concrete masonry materials shall be in accordance with Section 2103.1. Clay or shale masonry units shall be in accordance with Section 2103.2.
2. The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).
3. Columns shall be reinforced with not less than one No. 4 bar centered in the column.
4. Columns shall be grouted solid.
5. Columns shall not exceed 12 feet (3658 mm) in height.
6. Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16.
7. Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars may be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.

2107.2.3 ACI 530/ASCE 5/TMS 402, Section 2.1.8.6.1.1, lap splices. The minimum length of lap splices for reinforcing bars in tension or compression, l_{ld}, shall be calculated by Equation 21-2, but shall not be less than 15 inches (380 mm).

$$l_{ld} = \frac{0.16 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \qquad \textbf{(Equation 21-2)}$$

For SI: $$l_{ld} = \frac{1.95 d_b^2 f_y \gamma}{K\sqrt{f'_m}}$$

where:

d_b = Diameter of reinforcement, inches (mm).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

l_{ld} = Minimum lap splice length, inches (mm).

K = The lesser of the masonry cover, clear spacing between adjacent reinforcement, or 5 times d_b, inches (mm).

γ = 1.0 for No. 3 through No. 5 reinforcing bars. 1.4 for No. 6 and No. 7 reinforcing bars. 1.5 for No. 8 through No. 9 reinforcing bars.

2107.2.4 ACI 530/ASCE 5/TMS 402, maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

2107.2.5 ACI 530/ASCE 5/TMS 402, splice for large bars. Reinforcing bars larger than No. 9 in size shall be spliced using mechanical connectors in accordance with ACI 530/ASCE 5/TMS 402, Section 2.1.8.6.3.

SECTION 2108
STRENGTH DESIGN OF MASONRY

2108.1 General. The design of hollow-unit clay and concrete masonry structures using strength design shall comply with the provisions of Sections 2106 and 2108.

Exception: Two-wythe solid-unit masonry is permitted to be used under Section 2108.9.

2108.2 Deleted.

2108.3 Required strength. The required strength shall be determined in accordance with the factored load combinations of Section 1605.2. Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The required moment, M_u, shall include the moment induced by relative lateral displacement.

2108.4 Design strength. Design strength is the nominal strength, multiplied by the strength-reduction factor, ϕ, as specified in this section. Masonry members shall be proportioned such that the design strength exceeds the required strength.

The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength of the member, except that the nominal shear strength need not exceed 2.5 times V_u.

2108.4.1 Beams, piers and columns.

2108.4.1.1 Flexure. Flexure with or without axial load, the value of ϕ shall be determined from Equation 21-3.

$$\phi = 0.8 - \frac{P_u}{A_e f'_m} \qquad \textbf{(Equation 21-3)}$$

and $0.60 \le \phi \le 0.80$

where:

A_e = Effective area of masonry, square inches(mm^2).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_u = Factored axial strength due to factored loads, pounds (N).

2108.4.1.2 Shear. Shear: $\phi = 0.80$.

2108.4.2 Wall design for out-of-plane loads.

2108.4.2.1 Walls with factored axial load stress of 0.05 f'_m or less. Flexure: $\phi = 0.80$. Shear: $\phi = 0.80$.

2108.4.2.2 Walls with factored axial load stress greater than 0.05f'_m. Axial load, axial load with flexure and flexure: $\phi = 0.80$. Shear: $\phi = 0.80$.

2108.4.3 Wall design for in-plane loads.

2108.4.3.1 Axial load and flexure. Axial load, axial load with flexure and flexure: $\phi = 0.65$.

For walls with symmetrical reinforcement in which f_y does not exceed 60,000 psi (414 MPa), the value ϕ may be increased linearly to 0.85 as the value of ϕP_n decreases from $0.10 f'_m A_e$ or $0.25\, P_b$ to zero.

For solid grouted walls, the value of P_b may be calculated by Equation 21-4.

$$P_b = 0.85 f'_m b a_b \qquad \textbf{(Equation 21-4)}$$

$$a_b = 0.85d\,\{e_{mu}/[e_{mu} + (f_y/E_s)]\} \qquad \textbf{(Equation 21-5)}$$

where:

a_b = Depth of equivalent rectangular stress block at balanced strain conditions, inches (mm).

b = Effective width of rectangular member or width of flange for T and I sections, inches (mm).

E_s = Modulus of elasticity of steel, psi (GPa).

e_{mu} = Maximum usable compressive strain of masonry.

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_b = Nominal balanced design axial strength, pounds (N).

2108.4.3.2 Shear. Shear: $\phi = 0.80$ where the design shear strength, ϕV_n, corresponds to the development of 1.25 times the nominal flexural strength of the wall. For all other cases, ϕ shall be equal to 0.60.

2108.4.4 Moment-resisting wall frames.

2108.4.4.1 Flexure with or without axial load. The value of ϕ shall be as determined from Equation 21-6; however, the value of ϕ shall not be less than 0.65 nor greater than 0.85.

$$\phi = 0.85 - 2\left(\frac{P_u}{A_n f'_m}\right) \qquad \textbf{(Equation 21-6)}$$

where:

A_n = Net cross-sectional area of masonry, square inches (mm^2).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_u = Factored axial strength due to factored loads, pounds (N).

2108.4.4.2 Shear. Shear: $\phi = 0.80$.

2108.4.5 Anchor bolts.

2108.4.5.1 Tension headed anchor bolts. When the capacity of the masonry controls: $\phi = 0.5$. When the capacity of the anchor bolt steel controls: $\phi = 0.9$.

2108.4.5.2 Bent-bar anchor bolts. When the capacity of the masonry controls: $\phi = 0.5$. When the capacity of the anchor bolt steel controls: $\phi = 0.9$. When the capacity of the anchor pullout controls: $\phi = 0.65$.

2108.4.5.3 Shear. When the capacity of the masonry controls: $\phi = 0.5$. When the capacity of the anchor bolt steel controls: $\phi = 0.9$.

2108.4.6 Reinforcement.

2108.4.6.1 Development. Development: $\phi = 0.80$.

2108.4.6.2 Splices. Splices: $\phi = 0.80$.

2108.5 Deformation requirements.

2108.5.1 Story drifts. Masonry structures shall be designed so the design story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from Table 1617.3.

2108.5.1.1 Cantilever shear walls. Cantilever shear walls shall be proportioned such that the maximum displacement, δ_{max}, at Level n does not exceed $0.01h_n$.

2108.5.2 Deflection—plain masonry members. Deflection calculations for plain masonry members shall be based on uncracked section properties.

2108.5.3 Deflection—reinforced masonry. Deflection calculations for reinforced masonry members shall be based on cracked section properties. Deflection calculations are permitted to be based on an effective moment of inertia in accordance with the following:

$$I_{eff}=I_n\left(\frac{M_{cr}}{M_a}\right)^3+I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right]\le I_n \quad \textbf{(Equation 21-7)}$$

where:

I_{cr} = Gross, cracked moment of inertia of wall cross section, inches^4 (mm^4).

I_{eff} = Effective moment of inertia, inches^4 (mm^4).

I_n = Moment of inertia of the net cross-sectional area of a member, inches^4 (mm^4).

M_a = Maximum moment in member due to the applied loading for which deflection is computed, inch-pounds (N-mm).

M_{cr} = Nominal cracking moment strength in masonry, inch-pounds (N-mm).

For determining drift, the calculated deflection shall be multiplied by C_d as indicated in Table 1617.6.

2108.6 Headed and bent-bar anchor bolts. Embedded bolts shall be grouted in place with at least 1 inch (25 mm) of grout between the bolt and the masonry, except that $^1/_4$ inch (6.4 mm) bolts are permitted to be placed in bed joints which are at least $^1/_2$ inch (12.7 mm) in thickness.

The factored loads on embedded anchor bolts and headed studs shall not exceed the design strengths determined in this section. When anchors are embedded in a cracked tension zone of a member under gravity loads, the anchor strengths as governed by masonry breakout for all anchor bolts, and as governed by tensile pullout of bent-bar anchor bolts, shall be multiplied by 0.7.

2108.6.1 Axial strength of headed anchor bolts. The design axial strength, B_a, for headed anchor bolts embedded in masonry shall be the lesser of the value given by Equation 21-8 (strength governed by masonry breakout) or Equation 21-9 (strength governed by steel).

$$B_a=4\phi A_p\sqrt{f'_m} \quad \textbf{(Equation 21-8)}$$

For SI:

$$B_a=0.332\phi A_p\sqrt{f'_m}$$

$$B_a=\phi A_b f_y \quad \textbf{(Equation 21-9)}$$

where:

A_b = Cross-sectional area of anchor bolt, square inches (mm^2).

A_{pt} = Projected area of masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, square inches (mm^2).

B_a = Design axial strength of an anchor bolt, pounds (N).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

ϕ = Strength reduction factor; $\phi = 0.5$ for Equation 21-8 and $\phi = 0.9$ for Equation 21-9.

2108.6.1.1 Projected area of masonry. The area A_{pt} in Equation 21-8 shall be given by Equation 21-10.

$$A_{pt}=\pi l_b^2 \quad \textbf{(Equation 21-10)}$$

where:

A_{pt} = Projected area of masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, square inches (mm^2).

l_b = Effective embedment depth of anchor bolt, inches (mm).

Where the projected areas A_{pt} of adjacent headed anchor bolts overlap, the projected area A_{pt} of each bolt shall be reduced by one-half of the overlapping area. The portion of the projected area falling in an open cell, head joint, core or outside of the wall shall be deducted from the value of A_{pt} calculated using Equation 21-10.

2108.6.1.2 Effective embedment length of headed anchor bolts. The effective embedment length of a headed anchor bolt, l_b, shall be the length of the embedment measured perpendicular from the surface of the masonry to head of the anchor bolt.

2108.6.1.2.1 Minimum effective embedment length of headed anchor bolts. The minimum effective embedment length of headed anchor bolts resisting axial forces shall be 4 bolt diameters or 2 inches (51 mm), whichever is greater.

2108.6.2 Axial strength of bent-bar anchor bolts. The design axial strength, B_a, for bent-bar anchor bolts (J- or L-bolts) embedded in masonry shall be the least of Equation 21-11 (strength governed by masonry breakout), Equation 21-12 (strength governed by steel), and Equation 21-13 (strength governed by anchor pullout):

$$B_a=4\phi A_p\sqrt{f'_m} \quad \textbf{(Equation 21-11)}$$

For SI:

$$B_a=0.332\phi A_p\sqrt{f'_m}$$

$$B_a=\phi A_b f_y \quad \textbf{(Equation 21-12)}$$

$$B_a = \phi(1.5f'_m e d_b + 300\pi(l_b + e + d_b)d_b) \quad \textbf{(Equation 21-13)}$$

The second term in Equation 21-13 shall be included only if continuous special inspection is provided during placement.

where:

A_b = Cross-sectional area of anchor bolt, square inches (mm^2).

A_{pt} = Projected area of masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, square inches (mm^2).

B_a = Design axial strength of an anchor bolt, pounds (N).

d_b = Diameter of reinforcement, inch (mm).

e = Eccentricity of $P_{u\beta}$ inch (mm).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

l_b = Effective embedment length of bent-bar anchor bolt, inch (mm).

ϕ = Strength reduction factor.

ϕ = 0.5 for Equation 21-11.

ϕ = 0.9 for Equation 21-12.

ϕ = 0.65 for Equation 21-13.

2108.6.2.1 Projected area of masonry. The area A_{pt} in Equation 21-8 shall be given by Equation 21-14.

$$A_{pt} = \pi l_b^2 \quad \textbf{(Equation 21-14)}$$

where:

A_{pt} = Projected area of masonry surface of a right circular cone for calculating tensile breakout capacity of anchor bolts, square inches (mm^2).

l_b = Effective embedment depth of anchor bolt, inches (mm).

Where the projected areas, A_{pt}, of adjacent bent-bar anchor bolts overlap, the projected area, A_{pt}, of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area falling in an open cell, head joint, core or outside of the wall shall be deducted from the value of A_{pt} calculated using Equation 21-14.

2108.6.2.2 Effective embedment length of bent-bar anchor bolts. The effective embedment of a bent-bar anchor bolt, l_b, shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end, minus 1 anchor bolt diameter.

2108.6.2.2.1 Minimum effective embedment length of bent-bar anchor bolts. The minimum effective embedment length of bent-bar anchor bolts resisting axial forces shall be 4 bolt diameters or 2 inches (51 mm), whichever is greater.

2108.6.3 Shear strength of headed and bent-bar anchor bolts. Where the anchor bolt edge distance, l_{be}, equals or exceeds 12 bolt diameters, the design shear strength, (B_v), shall be the lesser of the values given by Equation 21-15 (strength governed by masonry), or Equation 21-16 (strength governed by steel).

$$B_v = 4\phi A_{pv}\sqrt{f'_m} \quad \textbf{(Equation 21-15)}$$

For SI:

$$B_v = 0.332\phi A_{pv}\sqrt{f'_m}$$

$$B_v = \phi 0.6 A_b f_y \quad \textbf{(Equation 21-16)}$$

where:

A_b = Cross-sectional area of anchor bolt, square inches (mm^2).

A_{pv} = Projected area of masonry surface of one-half right circular cone for calculating shear breakout capacity of anchor bolts, square inches (mm^2).

B_v = Design shear strength of an anchor bolt, pounds (N).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

ϕ = Strength reduction factor; ϕ = 0.5 for Equation 21-15; ϕ = 0.9 for Equation 21-16.

Where the anchor bolt edge distance, l_{be}, is less than 12 bolt diameters, the value of B_v in Equation 21-15 shall be reduced by linear interpolation to zero at an l_{be} distance of 1 inch (25 mm).

2108.6.3.1 Projected area of masonry. The area A_{pv} shall be given by Equation 21-17.

$$A_{pv} = \pi l_{be}^2/2 \quad \textbf{(Equation 21-17)}$$

where:

A_{pv} = Projected area of masonry surface of one-half right circular cone for calculating shear breakout capacity of anchor bolts, square inches (mm^2).

l_{be} = Anchor bolt edge distance, the least distance measured from edge of masonry to surface of anchor bolt, inches (mm).

Where the projected areas A_{pv} of adjacent anchor bolts overlap, the projected area A_{pv} of each bolt shall be reduced by one-half of the overlapping area. The portion of the projected area falling in an open cell, head joint, core or outside of the wall shall be deducted from the value A_{pv} calculated using Equation 21-17.

2108.6.3.2 Minimum effective embedment length. The minimum effective embedment length of headed or bent-bar anchor bolts resisting shear forces shall be 4 bolt diameters or 2 inches (51 mm), whichever is greater.

2108.6.4 Combined axial and shear strength of anchor bolts. Anchor bolts subjected to combined shear and tension shall be designed to satisfy Equation 21-18.

$$\frac{b_a}{B_a}+\frac{b_v}{B_v}\leq 1 \qquad \textbf{(Equation 21-18)}$$

where:

B_a = Design axial strength of an anchor bolt, pounds (N).

B_v = Design shear strength of an anchor bolt, pounds (N).

b_a = Factored axial force on an anchor bolt, pounds (N).

b_v = Factored shear force on an anchor bolt, pounds (N).

2108.6.5 Anchor bolt placement. Anchor bolts shall be placed so as to meet the edge distance, embedment depth and spacing requirements of ACI 530/ASCE 5/TMS 402.

2108.7 Properties of materials.

2108.7.1 Modulus of elasticity of steel reinforcement. Unless otherwise determined by test, steel reinforcement modulus of elasticity (E_s) shall be taken to be 29,000,000 psi (200 GPa).

2108.7.2 Modulus of elasticity of masonry. The design of clay and concrete masonry shall be based on the following modulus of elasticity values:

E_m = $700f'_m$ for clay masonry

E_m = $900f'_m$ for concrete masonry

or the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism determined by test in accordance with the prism test method, Article 1.4B.3 of ACI 530.1/ASCE 6/TMS 602, and ASTM E 111.

where:

E_m = Modulus of elasticity of masonry, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

2108.7.3 Modulus of rigidity of masonry. The modulus of rigidity of masonry, E_v, shall be taken equal to 0.4 times the modulus of elasticity of masonry, E_m.

2108.7.4 Masonry compressive strength.

2108.7.4.1 Minimum compressive strength. Except for architectural components of masonry, the specified compressive strength of masonry, f'_m, shall equal or exceed 1,500 psi (10.34 MPa).

2108.7.4.2 Maximum compressive strength. The value of f'_m used to determine nominal strength values in this chapter shall not exceed 4,000 psi (27.58 MPa) for concrete masonry and shall not exceed 6,000 psi (41.37 MPa) for clay masonry.

2108.7.5 Modulus of rupture.

2108.7.5.1 Out-of-plane bending. The modulus of rupture, f_r, for plain masonry elements subjected to out-of-plane bending shall be taken from Table 2108.7.5.

For reinforced masonry, the modulus of rupture, f_r, used in calculating service load deflections shall be as follows:

1. For two-wythe brick masonry.

$$f_r = 2.0\sqrt{f'_m},\ 125 \text{ psi maximum.} \qquad \textbf{(Equation 21-19)}$$

For SI:

$$f_r = 0.166\sqrt{f'_m},\ 861 \text{ kPa maximum.}$$

2. For partially grouted hollow unit masonry.

$$f_r = 2.5\sqrt{f'_m},\ 125 \text{ psi maximum.} \qquad \textbf{(Equation 21-20)}$$

TABLE 2108.7.5
MODULUS OF RUPTURE FOR OUT-OF-PLANE BENDING (f_r)[a]

MASONRY TYPE	MORTAR TYPES (psi)			
	Portland cement/lime or mortar cement		Masonry cement and air-entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints in running or stack bond:				
Solid units	80	60	48	30
Hollow units[a]				
Ungrouted	50	38	30	18
Fully grouted	136	116	82	58
Parallel to bed joints in running bond:				
Solid units	160	120	96	60
Hollow units				
Ungrouted and partially grouted	100	76	60	38
Fully grouted	160	120	96	60
In stack bond	0	0	0	0

For SI: 1 pound per square inch = 0.00689 MPa.

a. For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between hollow units that are fully grouted and hollow units that are ungrouted based on amount (percentage) of grouting.

For SI:

$f_r = 0.21\sqrt{f'_m}$, 861 kPa maximum.

3. For fully grouted hollow unit masonry.

$f_r = 4.0\sqrt{f'_m}$, 235 psi maximum. **(Equation 21-21)**

For SI:

$f_r = 0.33\sqrt{f'_m}$, 1.6 MPa maximum.

2108.7.5.2 In-plane bending. The modulus of rupture, f_r, normal to bed joints for masonry elements subjected to in-plane forces shall be taken as 250 psi (1724 kPa). For grouted stack bond masonry, tension normal to the bed joints for in-plane bending shall be assumed to be resisted only by the continuous grout core section.

2108.7.6 Reinforcement strength. Masonry design shall be based on a reinforcement strength equal to the specified yield strength of reinforcement, f_y, that shall not exceed 60,000 psi (414 MPa).

2108.8 Section properties. Member strength shall be computed using section properties based on the minimum net cross-sectional area of the bedded and grouted cores of the member under consideration. Section properties shall be based on specified dimensions.

2108.9 Reinforced masonry. The requirements of this section are in addition to the requirements of Section 2106 and Sections 2108.1 through 2108.8, and govern masonry in which reinforcement is used to resist forces.

2108.9.1 Design assumptions. The following assumptions apply:

1. Masonry carries no tensile stress greater than the modulus of rupture.
2. Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material.
3. Nominal strength of singly reinforced masonry wall cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains. Strain in reinforcement and masonry walls shall be assumed to be directly proportional to the distance from the neutral axis.
4. The maximum usable strain, e_{mu}, at the extreme masonry compression fiber shall be assumed to be 0.0035 inch/inch (mm/mm) for clay masonry and 0.0025 inch/inch (mm/mm) for concrete masonry.
5. Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.
6. Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.
7. Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except when computing requirements for deflection.
8. Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular.
9. Masonry stress of 0.85 f'_m shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fiber of maximum compressive strain. Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

2108.9.2 Reinforcement requirements and details.

2108.9.2.1 Reinforcing bar size. Reinforcing bars used in masonry shall not be larger than a No. 11 bar. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-fourth of the least dimension of the cell, course or collar joint in which it is placed. The area of reinforcing bars placed in a cell, or in a course, of hollow unit construction shall not exceed 4 percent, except where splices occur, of the cell area. No more than two bars shall be placed in a cell of a wall or a wall frame.

2108.9.2.2 Joint reinforcement. Longitudinal and cross wire of joint reinforcement shall be a minimum W1.1, and shall not exceed one-half the joint thickness.

Joint reinforcement shall not be used to satisfy the minimum shear reinforcing area requirements in members in Seismic Design Categories D, E and F.

2108.9.2.3 Clear distance between parallel bars. The clear distance between parallel reinforcing bars shall not be less than the nominal diameter of the bars nor less than 1 inch (25.4 mm).

2108.9.2.4 Clear distance between vertical bars in columns and piers. In columns and piers, the clear distance between vertical-reinforcing bars shall not be less than one and one-half times the nominal bar diameter, nor less than $1^1/_2$ inches (38 mm).

2108.9.2.5 Clear distance between spliced bars. The clear distance limitations between reinforcing bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

2108.9.2.6 Bundling of reinforcing bars. Reinforcing bars shall not be bundled.

2108.9.2.7 Reinforcing bar cover. Reinforcing bars shall have a minimum masonry cover not less than $2^1/_2\, d_b$ nor less than the following:

1. Where the masonry face is exposed to earth or weather, 2 inches (51 mm) for bars larger than No. 5 and $1^1/_2$ inches (38 mm) for No. 5 bar or smaller.
2. Where the masonry is not exposed to earth or weather, $1^1/_2$ inches (38 mm).

2108.9.2.8 Standard hooks. A standard hook shall be one of the following:

1. A 180-degree (3.14 rad) turn plus an extension of at least 4 bar diameters, but not less than $2^1/_2$ inches (63 mm) at the free end of the bar.
2. A 135-degree (2.56 rad) turn plus an extension of at least 6 bar diameters at the free end of the bar.
3. A 90-degree (1.57 rad) turn plus an extension of at least 12 bar diameters at the free end of the bar.
4. For stirrup and tie anchorage only, either a 135-degree or a 180-degree (2.56 rad or 3.14 rad) turn plus an extension of at least 6 bar diameters at the free end of the bar.
5. The equivalent embedment length for standard hooks in tension, l_{dh}, shall be as follows:

$$l_{dh} = 13d_b \qquad \textbf{(Equation 21-22)}$$

where:

d_b = Diameter of reinforcement, inches (mm).

l_{dh} = Equivalent development length for a standard hook, inches (mm).

2108.9.2.9 Minimum bend diameter for reinforcing bars. The diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than values specified in Table 2108.9.2.9.

TABLE 2108.9.2.9
MINIMUM DIAMETERS OF BEND

BAR SIZE	GRADE	MINIMUM BEND
No. 3 thru No. 7	40	5 bar diameter
No. 3 thru No. 8	50 or 60	6 bar diameter
No. 9	50 or 60	8 bar diameter

2108.9.2.10 Development. The calculated tension or compression reinforcement shall be developed in accordance with the following provisions:

1. The required embedment length of reinforcement shall be determined by Equation 21-23.

$$l_d = l_{de}/\phi \qquad \textbf{(Equation 21-23)}$$

where:

$$l_{de} = \frac{0.13 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}} \qquad \textbf{(Equation 21-24)}$$

For SI:

$$l_{de} = \frac{1.56 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}}$$

where:

d_b = Diameter of reinforcement, inches (mm).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

l_d = Required development length of reinforcement, inches (mm).

l_{de} = Embedment length of reinforcement, inches (mm).

K = The lesser of the masonry cover or clear spacing between adjacent reinforcement, inches (mm).

ϕ = Strength reduction factor; $\phi = 0.8$.

γ = 1.0 for No. 3 through No. 5 reinforcing bars. 1.4 for No. 6 and No. 7 reinforcing bars. 1.5 for No. 8 and No. 9 reinforcing bars.

2. K shall not exceed $5d_b$.
3. The embedment length shall not be less than 12 inches (305 mm) for reinforcing bars nor 6 inches (152 mm) for wire.

2108.9.2.11 Splices. Reinforcement splices shall comply with one of the following:

1. The minimum length of lap for bars shall be 15 inches (380 mm) or the length determined by Equation 21-25, whichever is greater.

$$l_d = l_{de}/\phi \qquad \textbf{(Equation 21-25)}$$

where:

l_d = Required development length of reinforcement, inches (mm).

l_{de} = Embedment length of reinforcement, inches (mm).

ϕ = Strength reduction factor; $\phi = 0.8$ for Equation 21-25.

Reinforcing bars larger than No. 9 in size shall be spliced using mechanical connectors in accordance with Section 2108.9.2.11, Item 3.

Bars spliced by noncontact lap splices shall be spaced transversely a distance not greater than one fifth the required length of lap nor more than 8 inches (203 mm).

2. A welded splice shall have the bars butted and welded to develop in tension 125 percent of the yield strength of the bar, f_y. Welding shall conform to AWS D1.4.
3. Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the yield strength of the bar, f_y.

2108.9.2.12 Development of shear reinforcement. Shear reinforcement shall extend the depth of the member less cover distances.

2108.9.2.12.1 Anchoring of stirrups. The ends of single leg or U-stirrups shall be anchored by one of the following means:

1. A standard hook plus an effective embedment of 0.5 times the development length, l_d. The ef-

fective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, and the start of the hook (point of tangency).

2. For No. 5 bar and D31 wire and small, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of l_d /3. The l_d /3 embedment of a stirrup leg shall be taken as the distance mid-depth of member, and the start of the hook (point of tangency).
3. Between the anchored ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

2108.9.2.12.2 Reinforcing bars for shear strength. Except at wall intersections, the end of a reinforcing bar needed to satisfy shear strength requirements in accordance with Section 2108.9.3.5.2 shall be bent around the edge vertical-reinforcing bar with a 180-degree hook. At wall intersections, reinforcing bars needed to satisfy shear strength requirements in accordance with Section 2108.9.3.5.2 shall be bent around the edge vertical bar with a 90-degree (1.57 rad) standard hook, and shall extend horizontally into the intersecting wall.

2108.9.2.13 Maximum reinforcement percentages. The ratio of reinforcement, ρ, shall not exceed that given by Method A or B below.

2108.9.2.13.1 Method A. Method A is permitted to be used where the story drift does not exceed 0.010 h_{sx} as given in Table 1617.3 and if the extreme compressive fiber strains are less than 0.0035 inch/inch (mm/mm) for clay masonry and 0.0025 inch/inch (mm/mm) for concrete masonry.

1. When walls are subjected to in-plane forces, and for columns and beams, the critical strain condition corresponds to a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y.
2. When walls are subjected to out-of-plane forces, the critical strain condition corresponds to a strain in the reinforcement equal to 1.3 times the strain associated with reinforcement yield stress, f_y.

The strain at the extreme compression fiber shall be assumed to be 0.0035 inch/inch (mm/mm) for clay masonry and 0.0025 inch/inch (mm/mm) for concrete masonry.

The calculation of the maximum reinforcement ratio shall include factored gravity axial loads. The stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The strength of the compressive zone shall be calculated as 80 percent of f'_m times 80 percent of the area of the compressive zone. Stress in reinforcement in the compression zone shall be based on a linear strain distribution.

2108.9.2.13.2 Method B. Method B is permitted to be used where the story drift does not exceed 0.013 h_{sx} as given in Table 1617.3.

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.002. The strain shall be determined using factored forces and R equal to 1.5.
2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Item 1 is greater than 0.002.
3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of No. 3 closed ties at a maximum spacing of 8 inches (203 mm) on center within the grouted core, or equivalent approved confinement, to develop an ultimate compressive strain of at least 0.006.
4. The maximum longitudinal reinforcement ratio shall not exceed $0.15 f'_m / f_y$.

2108.9.3 Design of beams, piers and columns. The requirements of this section are for the design of masonry beams, piers and columns. For computational purposes, the value of f'_m shall not exceed 4,000 psi (27.56 MPa).

2108.9.3.1 Design assumptions. Member design forces shall be based on an analysis that considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of all beams, piers and columns.

The effects of cracking on member stiffness shall be considered.

The drift ratio of piers and columns shall satisfy the limits specified in Chapter 16.

2108.9.3.2 Maximum reinforcement limits. The maximum reinforcement in beams, piers and columns shall be determined by Section 2108.9.2.13.

2108.9.3.3 Required strength. Except as required by Sections 2108.9.3.5 through 2108.9.3.11, the required strength shall be determined in accordance with Section 2108.3.

2108.9.3.4 Design strength. Design strength provided by beam, pier or column cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ, specified in Section 2108.4.

2108.9.3.5 Nominal strength.

2108.9.3.5.1 Nominal axial and flexural strength. The nominal axial strength, P_n, and the nominal flexural strength, M_n, of a cross section shall be determined in accordance with the design assumptions of Sections 2108.9.1 and 2108.9.3.1.

The maximum nominal axial compressive strength shall be determined in accordance with Equation 21-26.

$$P_n = 0.80[0.85f'_m(A_n - A_s) + f_y A_s] \quad \textbf{(Equation 21-26)}$$

where:

A_n = Net cross-sectional area of masonry, square inches (mm^2).

A_s = Effective cross-sectional area of reinforcement, square inches (mm^2).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_n = Nominal axial strength in masonry, pounds (N).

2108.9.3.5.2 Nominal shear strength. Nominal shear strength, V_n, shall be computed as follows:

$$V_n = V_m + V_s \quad \textbf{(Equation 21-27)}$$

For $M/Vd_v < 0.25$:

$$V_n = 6A_n\sqrt{f'_m} \quad \textbf{(Equation 21-28)}$$

For $M/Vd_v > 1.00$:

$$V_n = 4A_n\sqrt{f'_m} \quad \textbf{(Equation 21-29)}$$

where:

A_n = Net cross-sectional area of masonry, square inches (mm^2).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

V_m = Shear strength provided by masonry, pounds (N).

V_n = Nominal shear strength, pounds (N).

V_s = Shear strength provided by shear reinforcement, pounds (N).

Value of M/Vd_v between 0.25 and 1.0 is permitted to be interpolated.

2108.9.3.5.2.1 Nominal masonry shear strength. Shear strength, V_m, provided by the masonry shall be as follows:

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

(Equation 21-30)

For SI:

$$V_m = 0.83\left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right]A_n\sqrt{f'_m} + 0.25P$$

where:

M/Vd_v need not be taken greater than 1.0 and

where:

A_n = Net cross-sectional area of masonry, square inches (mm^2).

d_v = Length of member in direction of shear force, inches (mm).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

M = Moment on a masonry section due to unfactored load, inch-pound (N-mm).

P = Axial force on a masonry section due to unfactored loads, pounds (N).

V = Shear on a masonry section due to unfactored loads, pounds (N).

V_m = Shear strength provided by masonry, pounds (N).

2108.9.3.5.2.2 Nominal shear strength provided by reinforcement. Nominal shear strength, V_s, provided by reinforcement shall be as follows:

$$V_s = 0.5\left(\frac{A_v}{s}\right)f_y d_v \quad \textbf{(Equation 21-31)}$$

where:

A_v = Cross-sectional area of shear reinforcement, square inches (mm^2).

d_v = Length of member in direction of shear force, inch (mm).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

s = Spacing of stirrups or of bent bars in direction parallel to that of main reinforcement, inches (mm).

V_s = Shear strength provided by shear reinforcement, pounds (N).

2108.9.3.6 Reinforcement.

1. Where transverse reinforcement is required, the maximum spacing shall not exceed one-half the depth of the member nor 48 inches (1219 mm).
2. Flexural reinforcement shall be uniformly distributed throughout the depth of the element.
3. Flexural elements subjected to load reversals shall be symmetrically reinforced.
4. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength.
5. The maximum flexural reinforcement ratio shall be determined by Section 2108.9.2.13.
6. Lap splices shall comply with the provisions of Section 2108.9.2.11.
7. Welded splices and mechanical splices that develop at least 125 percent of the specified yield strength of a bar may be used for splicing the reinforcement. Not more than two longitudinal bars shall be spliced at a section. The distance between

splices of adjacent bars shall be at least 30 inches (762 mm) along the longitudinal axis.

8. Specified yield strength of reinforcement shall not exceed 60,000 psi (414 MPa). The actual yield strength based on mill tests shall not exceed 1.3 times the specified yield strength.

2108.9.3.7 Seismic design provisions. The lateral seismic load resistance in any line or story level shall be provided by shear walls or wall frames, or a combination of shear walls and wall frames. Shear walls and wall frames shall provide at least 80 percent of the lateral stiffness in any line or story level.

Exception: Where seismic loads are determined based on R not greater than 2 and where joints satisfy the provisions of Section 2108.9.6.9, the piers may be used to provide seismic load resistance.

2108.9.3.8 Dimensional limits. Dimensions shall be in accordance with the following:

1. **Beams.**
   - 1.1. The nominal width of a beam shall not be less than 6 inches (152 mm).
   - 1.2. The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.
   - 1.3. The nominal depth of a beam shall not be less than 8 inches (203 mm).
2. **Piers.**
   - 2.1. The nominal width of a pier shall not be less than 6 inches (152 mm) and shall not exceed 16 inches (406 mm).
   - 2.2. The distance between lateral supports of a pier shall not exceed 30 times the nominal width of the piers except as provided for in Item 2.3.
   - 2.3. When the distance between lateral supports of a pier exceeds 30 times the nominal width of the pier, the provisions of Section 2108.9.4 shall be used for design.
   - 2.4. The nominal length of a pier shall not be less than three times the nominal width of the pier. The nominal length of a pier shall not be greater than six times the nominal width of the pier. The clear height of a pier shall not exceed five times the nominal length of the pier.

     Exception: The length of a pier is permitted to be equal to the width of the pier where the factored axial force at the location of maximum moment is less than $0.05 f'_m A_g$.
3. **Columns.**
   - 3.1. The nominal width of a column shall not be less than 12 inches (305 mm).
   - 3.2. The distance between lateral supports of a column shall not exceed 30 times the nominal width of the column.
   - 3.3. The nominal length of a column shall not be less than 12 inches (305 mm) and not greater than three times the nominal width of the column.

2108.9.3.9 Beams.

2108.9.3.9.1 Scope. Members designed primarily to resist flexure shall comply with the requirements of this section. The factored axial compressive force on a beam shall not exceed $0.05 A_e f'_m$.

2108.9.3.9.2 Longitudinal reinforcement. The variation in the longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.

The nominal flexural strength of a beam shall not be less than 1.5 times the nominal cracking moment strength of the beam. The modulus of rupture, f_r, for this calculation shall be assumed to be 235 psi (1.62 MPa).

2108.9.3.9.3 Transverse reinforcement. Transverse reinforcement shall be provided where V_u exceeds ϕV_m. Required shear, V_u, shall include the effects of drift. The value of V_u shall be based on δ_x. Alternatively, the value of V_u is permitted to be determined from that shear which will result from developing the plastic moment strength at each end of the member. The plastic moment strength shall be based on 1.25 times the specified yield strength of the reinforcing and a $\phi = 1.0$. When transverse shear reinforcement is required, the following provisions shall apply:

1. Shear reinforcement shall be a single bar with a 180-degree (3.14 rad) hook at each end.
2. Shear reinforcement shall be hooked around the longitudinal reinforcement.
3. The minimum transverse shear reinforcement ratio shall be 0.0007.
4. The first transverse bar shall not be more than one-fourth of the beam depth from the end of the beam.

2108.9.3.9.4 Construction. Beams shall be solid grouted.

2108.9.3.10 Piers.

2108.9.3.10.1 Scope. Piers proportioned to resist flexure and shear in conjunction with axial load shall comply with the requirements of this section. The factored axial compression on the piers shall not exceed $0.3 A_e f'_m$.

2108.9.3.10.2 Longitudinal reinforcement. A pier subjected to in-plane stress reversals shall be longitudinally reinforced symmetrically on both sides of the neutral axis of the pier.

1. One bar shall be provided in the end cells.

2. The nominal flexural strength of a pier shall not be less than 1.5 times the nominal cracking moment strength of the pier. The modulus of rupture, f_r for this calculation shall be assumed to be 235 psi (1.62 MPa).

2108.9.3.10.3 Transverse reinforcement. Transverse reinforcement shall be provided where V_u exceeds V_m. Required shear, V_u, shall include the effects of drift (δ_x). Alternatively, the value of V_u is permitted to be determined from that shear which will result from developing the plastic moment strength at each end of the member. The plastic moment strength shall be based on 1.25 times the specified yield strength of the reinforcing and $\phi = 1.0$.

2108.9.3.11 Columns.

2108.9.3.11.1 Scope. Columns shall comply with the requirements of this section.

2108.2.3.11.2 Longitudinal reinforcement. Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column.

1. Maximum reinforcement area shall be $0.03\,A_e$.
2. Minimum reinforcement area shall be $0.005\,A_e$.

2108.9.3.11.3 Lateral ties. Lateral ties shall be provided in accordance with Section 2106. Minimum lateral reinforcement area shall be $0.0018\,A_g$.

2108.9.3.11.4 Construction. Columns shall be solid grouted.

2108.9.4 Wall design for out-of-plane loads.

2108.9.4.1 General. The requirements of this section are for the design of walls for out-of-plane loads.

2108.9.4.2 Maximum reinforcement. The maximum reinforcement ratio shall be determined by Section 2108.9.2.13.

2108.9.4.3 Moment and deflection calculations. Moment and deflection calculations in Section 2108.9.4 are based on simple support conditions top and bottom. Other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

2108.9.4.4 Walls with factored axial load stress of $0.05 f'_m$ or less. The procedures set forth in this section, which consider the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, shall be used when the factored vertical load stress at the location of maximum moment does not exceed $0.05\,f'_m$ as computed by Equation 21-32.

$$\left(\frac{P_{uw}+P_{uf}}{A_g}\right) \le 0.05 f'_m \qquad \textbf{(Equation 21-32)}$$

where:

A_g = Gross area of wall, square inches (mm²).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_{uf} = Factored load from tributary floor or roof area, pounds (N).

P_{uw} = Factored weight of wall tributary to section under consideration, pounds (N).

Walls shall have a minimum nominal thickness of 6 inches (152 mm).

Required moment and axial force shall be determined at the midheight of the wall and shall be used for design. The factored moment, M_u, at the midheight of the wall shall be determined by Equation 21-33.

$$M_u = \frac{W_u h^2}{8} + P_{uf}\frac{e}{2} + P_u \Delta_u \qquad \textbf{(Equation 21-33)}$$

where:

$$P_u = P_{uw} + P_{uf} \qquad \textbf{(Equation 21-34)}$$

where:

e = Eccentricity of P_{uf}, inches (mm).

h = Effective height of a column, pilaster or wall, inches (mm).

M_u = Factored moment, inch-pounds (N-mm).

P_u = Factored axial strength due to factored loads, pounds (N).

P_{uf} = Factored load from tributary floor or roof loads, pounds (N).

P_{uw} = Factored weight of wall tributary to section under consideration, pounds (N).

Δ_u = Deflection due to factored loads, inches (mm).

The design strength for out-of-plane wall loading shall be determined by Equation 21-35.

$$M_u \le \phi M_n \qquad \textbf{(Equation 21-35)}$$

where:

$$M_n = A_{se} f_y (d - a/2) \qquad \textbf{(Equation 21-36)}$$

$A_{se} = (A_s f_y + P_u) / f_y$; effective area of steel.

(Equation 21-37)

$a = (P_u + A_s f_y) / 0.85 f'_m\, b$; depth of stress block due to factored loads.

(Equation 21-38)

where:

A_s = Effective cross-sectional area of reinforcement, square inches (mm²).

A_{se} = Effective area of reinforcement, square inches (mm²).

a = Depth of equivalent rectangular stress block, inches (mm).

b = Effective width of rectangular member or width of flange for T and I sections, inches (mm).

d = Distance from compression face of flexural member to centroid of longitudinal tensile reinforcement, inches (mm).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

M_n = Nominal moment strength, inch-pounds (N-mm).

P_u = Factored axial strength due to factored loads, pounds (N).

The nominal shear strength shall be determined by Section 2108.9.3.5.2.

2108.9.4.5 Walls with factored axial load stress greater than 0.05f'_m. The procedures set forth in Section 2108.9.4.4 shall be used for the design of masonry walls when the factored vertical load stresses at the location of maximum moment exceed $0.05f'_m$ but are less than $0.2f'_m$ and the slenderness ratio h'/t does not exceed 30.

Design strength provided by the wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ, specified in Section 2108.4. Walls shall be proportioned so that the design strength exceeds the required strength.

The nominal shear strength shall be determined by Section 2108.9.3.5.2.

2108.9.4.6 Deflection design. The midheight deflection, Δ_s, under service lateral and vertical loads (without load factors) shall be limited by Equation 21-39.

$$\Delta_s = 0.007h \qquad \textbf{(Equation 21-39)}$$

where:

h = Effective height of a column, pilaster or wall, inches (mm).

$P\Delta$ effects shall be included in the deflection calculation. The midheight deflection shall be computed with Equation 21-40 or 21-41.

$$\Delta_s = \frac{5M_{ser}h^2}{48E_m I_g} \text{ for } M_{ser} \le M_{cr} \qquad \textbf{(Equation 21-40)}$$

$$\Delta_s = \frac{5M_{cr}h^2}{48E_m I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_m I_{cr}} \text{ for } M_{cr} < M_{ser} < M_u \qquad \textbf{(Equation 21-41)}$$

where:

E_m = Modulus of elasticity of masonry, psi (MPa).

h = Effective height of a column, pilaster or wall, inches (mm).

I_g, I_{cr} = Gross, cracked moment of inertia of wall cross section, inches^4 (mm^4).

M = Moment on a masonry section due to unfactored load, inch-pounds (N-mm).

M_{cr} = Nominal cracking moment strength in masonry, inch-pounds (N-mm).

M_{ser} = Service moment at midheight of panel, including $P\Delta$ effects, inch-pounds (N-mm).

M_u = Factored moment, inch-pounds (N-mm).

Δ_s = Horizontal deflection at midheight under service load, inches (mm).

The cracking moment strength of the wall shall be determined from the formula:

$$M_{cr} = Sf_r \qquad \textbf{(Equation 21-42)}$$

where:

f_r = Modulus of rupture, psi (MPa).

M_{cr} = Nominal cracking moment strength in masonry, inch-pounds (N-mm).

S = Uncracked section modulus, cubic inches (mm^3).

The modulus of rupture, f_r shall be determined from Section 2108.7.5.1, Equation 21-19, 21-20 or 21-21.

2108.9.5 Wall design for in-plane loads.

2108.9.5.1 General. The requirements of this section are for the design of walls for in-plane loads.

2108.9.5.2 Reinforcement. Reinforcement shall be in accordance with the following:

1. When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.5 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall from Equation 21-42.
2. The amount of vertical reinforcement shall not be less than one-half the horizontal reinforcement.
3. The maximum reinforcement ratio shall be determined by Section 2108.9.2.13.

2108.9.5.3 Design strength. Design strength provided by the shear wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ, specified in Section 2108.4.3.

2108.9.5.4 Axial strength. The nominal axial strength of the shear wall supporting axial loads only shall be calculated by Equation 21-43.

$$P_o = 0.85f'_m(A_e - A_s) + f_yA_s \qquad \textbf{(Equation 21-43)}$$

where:

A_e = Effective area of masonry, square inches (mm²).

A_s = Effective cross-sectional area of reinforcement, square inches (mm²).

f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

P_o = Nominal axial strength without bending, pounds (N).

Axial design strength provided by the shear wall cross section shall satisfy Equation 21-44.

$$P_u < \phi\, 0.80\, P_o \qquad \textbf{(Equation 21-44)}$$

where:

P_o = Nominal axial strength without bending, pounds (N).

P_u = Factored axial strength due to factored loads, pounds (N).

2108.9.5.5 Shear strength. Shear strength shall be determined by Section 2106.5.3.2 in Seismic Design Category D and Section 2108.4.

2108.9.6 Special masonry moment frames (wall frames).

2108.9.6.1 Calculations. The calculation of required strength of the members shall be in accordance with principles of engineering mechanics, and shall consider the effects of the relative stiffness degradation of the beams and columns.

2108.9.6.1.1 Yielding. Flexural yielding shall be limited to the beams at the face of the columns and to the bottom of the columns at the base of the structure.

2108.9.6.2 Reinforcement.

2108.9.6.2.1 Moment strength. The nominal moment strength at any section along a member shall not be less than one-half the higher moment strength provided at the two ends of the member.

2108.9.6.2.2 Lap splices. Lap splices are permitted only within the center half of the member length.

2108.9.6.2.3 Other splices. Welded splices and mechanical connections may be used for splicing the reinforcement at any section, provided not more than alternate longitudinal bars are spliced at a section, and the distance between splices on alternate bars is at least 24 inches (610 mm) along the longitudinal axis.

2108.9.6.2.4 Yield strength. Reinforcement shall have a specified yield strength of 60,000 psi (414 MPa). The actual yield strength shall not exceed 1.25 times the specified yield strength.

2108.9.6.3 Wall frame beams.

2108.9.6.3.1 Compression force. Factored axial compression force on the beam shall not exceed 0.10 times the net cross-sectional area of the beam, A_n, times the specified compressive strength, f'_m.

2108.9.6.3.2 Reinforcement ratio. Beams interconnecting vertical elements of the lateral-load resisting system shall be limited to a reinforcement ratio equal to the lesser of $0.15 f'_m/f_y$ or that determined in accordance with Section 2108.9.2.13. All reinforcement in the beam and adjacent to the beam in a reinforced concrete roof or floor system shall be used to calculate the reinforcement ratio.

2108.9.6.3.3 Minimum clear span. Clear span for the beam shall not be less than two times its depth.

2108.9.6.3.4 Beam depth. Nominal depth of the beam shall not be less than two units or 16 inches (406 mm), whichever is greater. The nominal depth to nominal width ratio shall not exceed 6.

2108.9.6.3.5 Beam width. Nominal width of the beams shall equal or exceed the following criteria:

1. Eight inches (203 mm),
2. Width required by Section 2108.9.3.8, Item 1, and
3. $^1/_{26}$ of the clear span between column faces.

2108.9.6.4 Longitudinal reinforcement for beams.

2108.9.6.4.1 Spacing. Longitudinal reinforcement shall not be spaced more than 8 inches (203 mm) on center.

2108.9.6.4.2 Distribution. Longitudinal reinforcement shall be uniformly distributed along the depth of the beam.

2108.9.6.4.3 Flexural strength. The nominal flexural strength of a beam shall not be less than 1.5 times the nominal cracking moment strength of the beam. The modulus of rupture, f_r, for this calculation shall be assumed to be 235 psi (1.62 MPa).

2108.9.6.4.4 Reinforcement for each masonry unit. At any section of a beam, each masonry unit through the beam depth shall contain longitudinal reinforcement.

2108.9.6.5 Transverse reinforcement for beams.

2108.9.6.5.1 Minimum shear strength. The shear V_u shall not be less than that which results from development of the plastic moment strength at each restrained end of the member and shall include the effects of gravity loads. The plastic moment strength shall be calculated using 1.25 times the specified yield strength of the reinforcing and ϕ of 1.0.

2108.9.6.5.2 Hooks. Transverse reinforcement shall be hooked around top and bottom longitudinal bars and shall be terminated with a standard 180-degree (3.14 rad) hook.

2108.9.6.5.3 End region spacing. Within an end region extending one beam depth from wall frame column faces and at any region at which beam plastic hinges may form during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one-fourth the nominal depth of the beam.

2108.9.6.5.4 Other maximum spacing. The maximum spacing of transverse reinforcement shall not exceed one-half the nominal depth of the beam or that required for shear strength.

2108.9.6.5.5 Reinforcement ratio. The minimum transverse reinforcement ratio shall be 0.0015.

2108.9.6.5.6 First bar location. The first transverse bar shall not be more than 4 inches (102 mm) from the face of the column.

2108.9.6.6 Wall frame columns.

2108.9.6.6.1 Maximum compression force. Factored axial compression force on the wall frame column shall not exceed 0.15 times the net cross-sectional area of the column, A_n, times the specified compressive strength, f'_m. The compressive stress shall also be limited by the maximum reinforcement ratio.

2108.9.6.6.2 Parallel column dimension. The nominal dimension of the column parallel to the plane of the wall frame shall not be less than two full units or 32 inches (813 mm), whichever is greater.

2108.9.6.6.3 Height-to-depth ratio. The nominal dimension of the column perpendicular to the plane of the wall frame shall not be less than 8 inches (203 mm) nor $^1/_{14}$ of the clear height between beam faces.

2108.9.6.6.4 Height-to-depth ratio. The clear height-to-depth ratio of column members shall not exceed 5.

2108.9.6.7 Longitudinal reinforcement for columns.

2108.9.6.7.1 Minimum number. A minimum of four longitudinal bars shall be provided at all sections of every wall frame column member.

2108.9.6.7.2 Distribution. The flexural reinforcement shall be uniformly distributed across the member depth.

2108.9.6.7.3 Moment strength. The nominal moment strength at any section along a member shall not be less than 1.5 times the cracking moment strength.

2108.9.6.7.4 Reinforcement ratio. Vertical reinforcement in wall-frame columns shall be limited to a maximum reinforcement ratio equal to the lesser of $0.15 f_m / f_y$ or that determined in accordance with Section 2108.9.2.13.

2108.9.6.8 Transverse reinforcement for columns.

2108.9.6.8.1 Minimum shear strength. The shear V_u shall not be less than that which results from development of the plastic moment strength at each restrained end of the member and shall include the effects of gravity loads. The plastic moment strength shall be calculated using 1.25 times the specified yield strength of the reinforcing and ϕ of 1.0.

2108.9.6.8.2 Hooks. Transverse reinforcement shall be hooked around the extreme longitudinal bars and shall be terminated with a standard 180-degree hook.

2108.9.6.8.3 Spacing. The spacing of transverse reinforcement shall not exceed one-fourth the nominal dimension of the column parallel to the plane of the wall frame.

2108.9.6.8.4 Reinforcement ratio. The minimum transverse reinforcement ratio shall be 0.0015.

2108.9.6.9 Wall frame beam-column intersection.

2108.9.6.9.1 Beam depth. Beam-column intersection dimensions in masonry wall frames shall be proportioned such that the wall frame column depth in the plane of the frame satisfies Equation 21-45.

$$h_p > \frac{4{,}800 d_{bb}}{\sqrt{f'_g}} \qquad \textbf{(Equation 21-45)}$$

For SI:

$$h_p > \frac{400 d_{bb}}{\sqrt{f'_g}} \qquad \textbf{(Equation 21-46)}$$

where:

h_p = Pier depth in the plane of the wall frame, inches (mm).

d_{bb} = Diameter of the largest beam longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, inches (mm).

f'_g = Specified compressive strength of grout, psi (MPa), shall not exceed 5,000 psi (34.48 MPa).

Beam depth in the plane of the frame shall satisfy Equation 21-47.

$$h_b > \frac{1{,}800 d_{bp}}{\sqrt{f'_g}} \qquad \textbf{(Equation 21-47)}$$

For SI:

$$h_b > \frac{150 d_{bp}}{\sqrt{f'_g}} \qquad \textbf{(Equation 21-48)}$$

where:

h_b = Beam depth in the plane of the wall frame, inches (mm).

d_{bp} = Diameter of the largest column (pier) longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, inches (mm).

f'_g = Specified compressive strength of grout, psi (MPa) shall not exceed 5,000 psi (34.48 MPa).

Nominal shear strength of beam-column intersections shall exceed the shear occurring when wall frame beams develop their nominal flexural strength.

2108.9.6.9.2 Reinforcement details. Beam longitudinal reinforcement terminating in a wall frame column shall be extended to the far face of the column and shall be anchored by a standard hook bent back into the wall frame column.

Special horizontal shear reinforcement crossing a potential diagonal beam column shear crack shall be provided such that:

$$A_s \geq \frac{0.5V_n}{f_y} \qquad \textbf{(Equation 21-49)}$$

where:

A_s = Cross-sectional area of reinforcement, inches (mm^2).

V_n = Nominal shear strength, pounds (N).

f_y = Specified yield strength of the reinforcement or the anchor bolt as applicable, psi (MPa).

Special horizontal shear reinforcement shall be anchored by a standard hook around the extreme wall frame column reinforcing bars.

Vertical shear forces can be considered to be carried by a combination of masonry shear-resisting mechanisms and truss mechanisms involving intermediate column reinforcing bars.

The nominal horizontal shear stress at the beam-column intersection shall not exceed the lesser of 350 psi (2.41 MPa) or:

$$7\sqrt{f'_m} \qquad \textbf{(Equation 21-50)}$$

For SI:

$$0.58\sqrt{f'_m}$$

2108.10 Design of plain (unreinforced) masonry members.

2108.10.1 General.

2108.10.1.1 Strength for resisting loads. Plain (unreinforced) masonry members shall be designed using the flexural tensile strength of masonry units. Mortar and grout to resist design loads is permitted.

2108.10.1.2 Strength contribution from reinforcement. Stresses in reinforcement shall not be considered effective in resisting design loads.

2108.10.1.3 Design criteria. Plain masonry members shall be designed to remain uncracked.

2108.10.2 Flexural strength of plain (unreinforced) masonry members. The following assumptions shall apply for the determination of the flexural strength of plain masonry members:

1. Strength design of members for flexure and axial load shall be in accordance with principles of engineering mechanics.
2. Strain in masonry shall be directly proportional to the distance from the neutral axis.
3. Flexural tension in masonry shall be assumed directly proportional to strain.
4. Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed directly proportional to strain. Maximum compressive stress shall not exceed $0.85f'_m$.

2108.10.3 Axial load strength of plain (unreinforced) masonry members. Design axial load strength shall be in accordance with Equation 21-51 or Equation 21-52.

$$\phi P_n = \phi A_n f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} < 99 \qquad \textbf{(Equation 21-51)}$$

$$\phi P_n = \phi A_n f'_m \left(\frac{70r}{h}\right)^2 \text{ for } \frac{h}{r} \geq 99 \qquad \textbf{(Equation 21-52)}$$

where:

A_n = Net cross-sectional area of masonry, square inches (mm^2).

f'_m = Specified compressive strength of masonry at age of 28 days, psi (MPa).

h = Effective height of a column, pilaster or wall, inches (mm).

P_n = Nominal axial strength in masonry, pounds (N).

r = Radius of gyration, inches (mm).

2108.10.4 Shear strength of plain (unreinforced) masonry members.

2108.10.4.1 Nominal shear strength. Nominal shear strength V_n shall be the lesser of the following:

1. $1.5\sqrt{f'_m}\,A_n$, pounds (For SI: $0.125\sqrt{f'_m}\,N$, where f'_m is in MPa and A_n is in mm^2).
2. $120A_n$, pounds (For SI: $0.83\,A_n$, N, where A_n is in mm^2).
3. $37A_n + 0.3N_v$ for running bond masonry not grouted solid, pounds (For SI: $0.26A_n + 0.3N_v$ where A_n is in mm^2 and N_v is in N).
4. $37A_n + 0.3N_v$ for stack bond masonry with open-end units and grouted solid, pounds (For SI: $0.26\,A_n + 0.3\,N_v$ where A_n is in mm^2 and N_v is in N).
5. $60A_n + 0.3N_v$ for running bond masonry grouted solid, pounds (For SI: $0.4A_n + 0.3N_v$ where A_n is in mm^2 and N_v is in N).
6. $15A_n$ for stack bond other than open-end units grouted solid, pounds (For SI: $0.103A_n + 0.3N_v$ where A_n is in mm^2 and N_v is in N).

SECTION 2109
EMPIRICAL DESIGN OF MASONRY

2109.1 [Comm 62.2109 (1)] General. Empirically designed masonry shall conform to this chapter or Chapters 1 and 5 of ACI 530/ASCE 5/TMS 402. Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of IBC Chapter 16.

2109.1.1 Limitations. Empirical masonry design shall not be utilized for any of the following conditions:

1. The design or construction of masonry in buildings assigned to Seismic Design Category D, E or F as specified in Section 1616, and the design of the lateral-force-resisting system for buildings assigned to Seismic Design Category B or C.
2. The design or construction of masonry structures located in areas where the 3-second gust wind speed from Figure 1609 exceeds 110 mph (145 km/hr).
3. Buildings more than 35 feet (10 668 mm) in height that have masonry wall lateral-force-resisting systems.

In buildings that exceed one or more of the above limitations, masonry shall be designed in accordance with the engineered design provisions of Section 2107 or Section 2108.

2109.2 Lateral stability.

2109.2.1 Shear walls. Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

2109.2.1.1 Shear wall thickness. Minimum nominal thickness of masonry shear walls shall be 8 inches (203 mm).

Exception: Shear walls of one-story buildings are permitted to be a minimum nominal thickness of 6 inches (152 mm).

2109.2.1.2 Cumulative length of shear walls. In each direction in which shear walls are required for lateral stability, the minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. Cumulative length of shear walls shall not include openings.

2109.2.1.3 Maximum diaphragm ratio. Masonry shear walls shall be provided so that the span to width or depth ratio of floor or roof diaphragms does not exceed that indicated in Table 2109.2.1.3.

TABLE 2109.2.1.3
MAXIMUM SPAN TO WIDTH (DEPTH) RATIO OF FLOOR AND ROOF DIAPHRAGMS

FLOOR OR ROOF DIAPHRAGMS CONSTRUCTION	MAXIMUM SPAN TO WIDTH (DEPTH) RATIO OF DIAPHRAGM
Cast-in-place concrete	5:1
Precast concrete	4:1
Metal deck with concrete fill	3:1
Metal deck with no fill	2:1
Wood diaphragm	2:1

2109.2.2 Roofs. The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.

2109.2.3 Surface-bonded walls. Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of this code for masonry wall construction, except where otherwise noted in this section.

2109.2.3.1 Strength. Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 2109.2.3.1. Allowable stresses not specified in Table 2109.2.3.1 shall comply with the requirements of the ACI 530/ASCE 5/TMS 402.

TABLE 2109.2.3.1
ALLOWABLE STRESS GROSS CROSS-SECTIONAL AREA FOR DRY-STACKED, SURFACE-BONDED CONCRETE MASONRY WALLS

DESCRIPTION	MAXIMUM ALLOWABLE STRESS (psi)
Compression standard block	45
Shear	10
Flexural tension	
Vertical span	18
Horizontal span	30

For SI: 1 pound per square inch = 0.006895 MPa.

2109.2.3.2 Construction. Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units. Mixing and application of mortar, and curing and protection shall comply with ASTM C 946.

2109.3 Compressive stress requirements.

2109.3.1 Vertical dead plus live loads. Compressive stresses in masonry due to vertical dead plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109.3.2.1. Dead and live loads shall be in accordance with Chapter 16, with live load reductions as permitted in Section 1607.9.

2109.3.2 Maximum values. The compressive stresses in masonry shall not exceed the values given in Table 2109.3.2. Stress shall be calculated based on actual rather than nominal dimensions.

2109.3.2.1 Calculated compressive stresses. Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.

2109.3.2.2 Multiwythe walls. The allowable stress shall be as given in Table 2109.3.2 for the weakest combination of the units used in each wythe.

2109.4 Lateral support.

2109.4.1 General. Masonry walls shall be laterally supported in either the horizontal or the vertical direction at intervals not exceeding those given in Table 2109.4.1.

TABLE 2109.3.2
ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY

CONSTRUCTION; COMPRESSIVE STRENGTH OF UNIT GROSS AREA (psi)	ALLOWABLE COMPRESSIVE STRESSES[a] GROSS CROSS-SECTIONAL AREA (psi)	
	Type M or S mortar	Type N mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8,000 or greater	350	300
4,500	225	200
2,500	160	140
1,500	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4,500 or greater	225	200
2,500	160	140
1,500	115	100
Solid masonry of solid concrete masonry units:		
3,000 or greater	225	200
2,000	160	140
1,200	115	100
Masonry of hollow load-bearing units:		
2,000 or greater	140	120
1,500	115	100
1,000	75	70
700	60	55
Hollow walls (noncomposite masonry bonded)[b]		
Solid units:		
2,500 or greater	160	140
1,500	115	100
Hollow units	75	70
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry		
Coursed, rough or random	120	100

For SI: 1 pound per square inch = 0.006895 MPa.

a. Linear interpolation for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table is permitted.

b. Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls unless collar joints are filled with mortar or grout.

TABLE 2109.4.1
WALL LATERAL SUPPORT REQUIREMENTS

CONSTRUCTION	MAXIMUM WALL LENGTH TO THICKNESS OR WALL HEIGHT TO THICKNESS
Bearing walls	
Solid units or fully grouted	20
All others	18
Nonbearing walls	
Exterior	18
Interior	36

Comm 62.2109(2) Openings. Unless evidence is provided to show that openings do not cause lateral stability and stress requirements to be exceeded, the amount of openings in a masonry wall shall not exceed the limits set forth in Table 62.2109-1.

2109.4.2 Thickness. Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes. For cantilever walls, except for parapets, the ratio of height to nominal thickness shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets, see Section 2109.5.5.

2109.4.3 Lateral support. Lateral support shall be provided by cross walls, pilasters, buttresses or structural frame members when the limiting distance is taken horizontally, or by floors, roofs acting as diaphragms, or structural frame members when the limiting distance is taken vertically.

TABLE 62.2109–1
MAXIMUM RATIO OF LATERALLY UNSUPPORTED HEIGHT OR LENGTH TO THICKNESS FOR EXTERIOR WALLS WITH OPENINGS[a]

TYPE OF MASONRY	PERCENT OF OPENINGS AT ANY HORIZONTAL PLANE OF WALL			
	20	40	60	Over 60
Single-wythe walls of solid or grouted walls of solid units	20	16	12	Submit design calculations
All other masonry	18	14	10	

For SI: 1 foot = 304.8 mm.
a. The percentage of openings shall be calculated for each 100 lineal feet of wall or portion thereof at any horizontal plane of wall.

2109.5 Thickness of masonry.

2109.5.1 Thickness of walls. The nominal thickness of masonry walls shall conform to the requirements of Section 2109.5.

2109.5.2 Minimum thickness. The minimum thickness of masonry bearing walls more than one story high shall be 8 inches (203 mm). Bearing walls of one story buildings shall not be less than 6 inches (152 mm) thick.

2109.5.3 Rubble stone walls. The minimum thickness of rough or random or coursed rubble stone walls shall be 16 inches (406 mm).

2109.5.4 Change in thickness. Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.

2109.5.5 Parapet walls.

2109.5.5.1 Minimum thickness. Unreinforced parapet walls shall be at least 8 inches (203 mm) thick, and their height shall not exceed three times their thickness.

2109.5.5.2 Additional provisions. Additional provisions for parapet walls are contained in Sections 1504.2, 1504.3 and 1504.4.

2109.5.6 Foundation walls. For the minimum thicknesses of masonry foundation walls, see Section 1805.5.

2109.6 Bond.

2109.6.1 General. The facing and backing of multiple wythe masonry walls shall be bonded in accordance with Section 2109.6.2, 2109.6.3 or 2109.6.4.

2109.6.2 Bonding with masonry headers.

2109.6.2.1 Solid units. Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 3 inches (76 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).

2109.6.2.2 Hollow units. Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units that are at least 50 percent greater in thickness than the units below.

2109.6.2.3 Masonry bonded hollow walls. In masonry bonded hollow walls, the facing and backing shall be so bonded so that not less than 4 percent of the wall surface of each face is composed of masonry bonded units extending not less than 3 inches (76 mm) into the backing. The distance between adjacent bonders shall not exceed 24 inches (610 mm) either vertically or horizontally.

2109.6.3 Bonding with wall ties or joint reinforcement.

2109.6.3.1 Bonding with wall ties. Except as required by Section 2109.6.3.1.1, where the facing and backing (adjacent wythes) of masonry walls are bonded with wire size W2.8 (4.8 mm) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each $4^1/_2$ square feet (0.42 m^2) of wall area. Ties in alternate courses shall be staggered. The maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls the ends of ties shall be bent to 90-degree (1.57 rad) angles to provide hooks no less than 2 inches (51 mm) long. Additional bonding ties shall be provided at all openings, spaced not more than 3 feet (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.

2109.6.3.1.1 Bonding with adjustable wall ties. Where the facing and backing (adjacent wythes) of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 1.77 square feet (0.164 m^2) of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 16 inches (406 mm). The maximum vertical offset of bed joints from one wythe to the other shall be $1^1/_4$ inches (32 mm). The maximum clearance between connecting parts of the ties shall be $^1/_{16}$ inch (1.6 mm). When pintle legs are used, ties shall have at least two wire size W2.8 (4.8 mm) legs.

2109.6.3.2 Bonding with prefabricated joint reinforcement. Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each $2^2/_3$ square feet (0.25 m^2) of wall area. The vertical spacing of the joint reinforcing shall not exceed 24 inches (610 mm). Cross wires on prefabricated joint reinforcement shall not be less than W1.7. The longitudinal wires shall be embedded in the mortar.

2109.6.4 Bonding with natural or cast stone.

2109.6.4.1 Ashlar masonry. In ashlar masonry, bonder units, uniformly distributed, shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 4 inches (102 mm) into the backing wall.

2109.6.4.2 Rubble stone masonry. Rubble stone masonry 24 inches (610 mm) or less in thickness shall have bonder units with a maximum spacing of 3 feet (914 mm) vertically and 3 feet (914 mm) horizontally, and if the masonry is of greater thickness than 24 inches (610 mm), shall have one bonder unit for each 6 square feet (0.56 m^2) of wall surface on both sides.

2109.6.5 Masonry bonding pattern.

2109.6.5.1 Masonry laid in running bond. Each wythe of masonry shall be laid in running bond, head joints in successive courses shall be offset by not less than one-fourth the unit length, or the masonry walls shall be reinforced longitudinally as required in Section 2109.6.5.2.

2109.6.5.2 Masonry laid in stack bond. Where unit masonry is laid with less head joint offset than in Section 2109.6.5.1, the minimum area of horizontal reinforcement placed in mortar bed joints or in bond beams spaced not more than 48 inches (1219 mm) apart, shall be 0.0003 times the vertical cross-sectional area of the wall.

2109.7 Anchorage.

2109.7.1 General. Masonry elements shall be anchored in accordance with Sections 2109.7.2 through 2109.7.4.

2109.7.2 Intersecting walls. Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the methods indicated in Sections 2109.7.2.1 through 2109.7.2.5.

2109.7.2.1 Bonding pattern. Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 3 inches (76 mm) on the unit below.

2109.7.2.2 Steel connectors. Walls shall be anchored by steel connectors having a minimum section of $^1/_4$ inch (6.4 mm) by $1^1/_2$ inches (38 mm), with ends bent up at least 2 inches (51 mm) or with cross pins to form anchorage. Such anchors shall be at least 24 inches (610 mm) long and the maximum spacing shall be 4 feet (1219 mm).

2109.7.2.3 Joint reinforcement. Walls shall be anchored by joint reinforcement spaced at a maximum distance of 8 inches (203 mm). Longitudinal wires of such reinforcement shall be at least wire size W1.7 and shall extend at least 30 inches (762 mm) in each direction at the intersection.

2109.7.2.4 Interior nonload-bearing walls. Interior nonload-bearing walls shall be anchored at their intersection, at vertical intervals of not more than 16 inches (406 mm) with joint reinforcement or $^1/_4$ inch (6.4 mm) mesh galvanized hardware cloth.

2109.7.2.5 Ties, joint reinforcement and anchors. Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.

2109.7.3 Floor and roof anchorage. Floor and roof diaphragms providing lateral support to masonry shall comply with Section 1607.3 and shall be connected to the masonry in accordance with Sections 2109.7.3.1 through 2109.7.3.3.

2109.7.3.1 Wood floor joists. Wood floor joists bearing on masonry walls shall be anchored to the wall at intervals not to exceed 6 feet (1829 mm) by metal strap anchors. Joists parallel to the wall shall be anchored with metal straps spaced not more than 6 feet (1829 mm) on centers extending over or under and secured to at least three joists. Blocking shall be provided between joists at each strap anchor.

2109.7.3.2 Steel floor joists. Steel floor joists bearing on masonry walls shall be anchored to the wall with $^3/_8$-inch (9.5 mm) round bars, or their equivalent, spaced not more than 6 feet (1829 mm) on center. Where joists are parallel to the wall, anchors shall be located at joist bridging.

2109.7.3.3 Roof diaphragms. Roof diaphragms shall be anchored to masonry walls with $^1/_2$-inch-diameter (12.7 mm) bolts 6 feet (1829 mm) on center or their equivalent. Bolts shall extend and be embedded at least 15 inches (381 mm) into the masonry, or be hooked or welded to not less than 0.20 square inch (129 mm^2) of bond beam reinforcement placed not less than 6 inches (152 mm) from the top of the wall.

2109.7.4 Walls adjoining structural framing. Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of $^1/_2$-inch (12.7 mm) bolts spaced at 4 feet (1219 mm) on center embedded 4 inches (102 mm) into the masonry, or their equivalent area.

2109.8 Adobe construction. Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction.

2109.8.1 Unstabilized adobe.

2109.8.1.1 Compressive strength. Adobe units shall have an average compressive strength of 300 psi (2068 kPa) when tested in accordance with ASTM C 67. Five samples shall be tested and no individual unit may have a compressive strength of less than 250 psi (1724 kPa).

2109.8.1.2 Modulus of rupture. Adobe units shall have an average modulus of rupture of 50 psi (345 kPa) when tested in accordance with the following procedure. Five samples shall be tested and no individual unit shall have a modulus of rupture of less than 35 psi (241 kPa).

2109.8.1.2.1 Support conditions. A cured unit shall be simply supported by 2-inch-diameter (51 mm) cylindrical supports located 2 inches (51 mm) in from each end and extending the full width of the unit.

2109.8.1.2.2 Loading conditions. A 2-inch-diameter (51 mm) cylinder shall be placed at midspan parallel to the supports.

2109.8.1.2.3 Testing procedure. A vertical load shall be applied to the cylinder at the rate of 500 pounds per minute (37 N/s) until failure occurs.

2109.8.1.2.4 Modulus of rupture determination. The modulus of rupture shall be determined by the formula:

$$f_r = 3WL_s/2bt^2 \qquad \textbf{(Equation 21-53)}$$

where, for the purposes of this section only:

b = Width of the test specimen measured parallel to the loading cylinder, inches (mm).

f_r = Modulus of rupture, psi (MPa).

L_s = Distance between supports, inches (mm).

t = Thickness of the test specimen measured parallel to the direction of load, inches (mm).

W = The applied load at failure, pounds (N).

2109.8.1.3 Moisture content requirements. Adobe units shall have a moisture content not exceeding 4 percent by weight.

2109.8.1.4 Shrinkage cracks. Adobe units shall not contain more than three shrinkage cracks and any single shrinkage crack shall not exceed 3 inches (76 mm) in length or $^1/_8$ inch (3.2 mm) in width.

2109.8.2 Stabilized adobe.

2109.8.2.1 Material requirements. Stabilized adobe shall comply with the material requirements of unstabilized adobe in addition to Sections 2109.8.2.1.1 and 2109.8.2.1.2.

2109.8.2.1.1 Soil requirements. Soil used for stabilized adobe units shall be chemically compatible with the stabilizing material.

2109.8.2.1.2 Absorption requirements. A 4-inch (102 mm) cube, cut from a stabilized adobe unit dried to a constant weight in a ventilated oven at 212°F to 239°F (100°C to 115°C), shall not absorb more than $2^1/_2$ percent moisture by weight when placed upon a constantly water-saturated, porous surface for 7 days. A minimum of five specimens shall be tested and each specimen shall be cut from a separate unit.

2109.8.3 Working stress. The allowable compressive stress based on gross cross-sectional area of adobe shall not exceed 30 psi (207 kPa).

2109.8.3.1 Bolts. Bolt values shall not exceed those set forth in Table 2109.8.3.1.

TABLE 2109.8.3.1
ALLOWABLE SHEAR ON BOLTS IN ADOBE MASONRY

DIAMETER OF BOLTS (inches)	MINIMUM EMBEDMENT (inches)	SHEAR (pounds)
$^1/_2$	—	—
$^5/_8$	12	200
$^3/_4$	15	300
$^7/_8$	18	400
1	21	500
$1^1/_8$	24	600

For SI: 1 inch = 25.4 mm, 1 pound = 4.448 N.

2109.8.4 Construction.

2109.8.4.1 General.

2109.8.4.1.1 Height restrictions. Adobe construction shall be limited to buildings not exceeding one story, except that two-story construction is allowed when designed by a registered design professional.

2109.8.4.1.2 Mortar restrictions. Mortar for stabilized adobe units shall comply with Chapter 21 or adobe soil. Adobe soil used as mortar shall comply with material requirements for stabilized adobe. Mortar for unstabilized adobe shall be portland cement mortar.

2109.8.4.1.3 Mortar joints. Adobe units shall be laid with full head and bed joints and shall be laid in full running bond.

2109.8.4.1.4 Parapet walls. Parapet walls constructed of adobe units shall be waterproofed.

2109.8.4.2 Wall thickness. The minimum thickness of exterior walls in one-story buildings shall be 10 inches (254 mm). The walls shall be laterally supported at intervals not exceeding 24 feet (7315 mm). The minimum thickness of interior bearing walls shall be 8 inches (203 mm). In no case shall the unsupported height of any wall constructed of adobe units exceed 10 times the thickness of such wall.

2109.8.4.3 Foundations.

2109.8.4.3.1 Foundation support. Walls and partitions constructed of adobe units shall be supported by foundations or footings that extend not less than 6 inches (152 mm) above adjacent ground surfaces and are constructed of solid masonry (excluding adobe) or concrete. Footings and foundations shall comply with Chapter 18.

2109.8.4.3.2 Lower course requirements. Stabilized adobe units shall be used in adobe walls for the first 4 inches (102 mm) above the finished first floor elevation.

2109.8.4.4 Isolated piers or columns. Adobe units shall not be used for isolated piers or columns in a load-bearing capacity. Walls less than 24 inches (610 mm) in length shall be considered isolated piers or columns.

2109.8.4.5 Tie beams. Exterior walls and interior bearing walls constructed of adobe units shall have a continuous tie beam at the level of the floor or roof bearing and meeting the following requirements.

2109.8.4.5.1 Concrete tie beams. Concrete tie beams shall be a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Concrete tie beams shall be continuously reinforced with a minimum of two No. 4 reinforcing bars. The ultimate compressive strength of concrete shall be at least 2,500 psi (17.2 MPa) at 28 days.

2109.8.4.5.2 Wood tie beams. Wood tie beams shall be solid or be built-up of lumber having a minimum nominal thickness of 1 inch (25 mm), and shall have a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Joints in wood tie beams shall be spliced a minimum of 6 inches (152 mm). No splices shall be allowed within 12 inches (305 mm) of an opening. Wood used in tie beams shall be approved naturally decay-resistant or pressure-treated wood.

2109.8.4.6 Exterior finish. Exterior walls constructed of unstabilized adobe units shall have their exterior surface covered with a minimum of two-coats of portland cement plaster having a minimum thickness of $^3/_4$ inch (19.1 mm) and conforming to ANSI A42.2. Lathing shall comply with ANSI A42.3. Fasteners shall be spaced at 16 inches (406 mm) on center maximum. Exposed wood surfaces shall be treated with an approved wood preservative or other protective coating prior to lath application.

2109.8.4.7 Lintels. Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of Chapter 16.

Comm 62.2109 (3) Jointing.

(a) **Expansion and shrinkage.** Joints commensurate with lateral stability requirements shall be installed in all exterior masonry to allow for expected growth of clay products and shrinkage of concrete products.

(b) **Vertical jointing.** Vertical movement joints shall be provided at a spacing in compliance with Table 62.2109–2.

(c) **Horizontal jointing.** Where supports such as shelf angles or plates are required to carry the weight of masonry above the foundation level, a pressure-relieving joint shall be provided between the structural support and any masonry that occurs below this level. The joint width shall be such as to prevent any load being transmitted from the support to any element directly below. All mortar and rigid materials shall be kept out of this joint. This type of joint shall be provided at all such supports in a concrete frame structure where clay masonry is exposed to the weather.

SECTION 2110 GLASS UNIT MASONRY

2110.1 Scope. This section covers the empirical requirements for nonload-bearing glass unit masonry elements in exterior or interior walls.

2110.1.1 Limitations. Solid or hollow approved glass block shall not be used in fire walls, party walls, fire separation assemblies or fire partitions, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls, or other approved joint materials. Wood strip framing shall not be used in fire separation assemblies that are required to be fire-resistance rated.

Exceptions:

1. Glass-block assemblies having a fire-resistance rating of not less than $^3/_4$ hour shall be permitted as opening protectives in fire separation assemblies or in fire partitions that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways or exit passageways.
2. Glass-block assemblies as permitted in Section 404.5, Exception 1.2.

TABLE 62.2109-2
MAXIMUM SPACING OF EXTERIOR MASONRY MOVEMENT JOINTS BETWEEN UNRESTRAINED ENDS[a] (feet)

		OPENINGS (PERCENT OF TOTAL WALL AREA)			
		0 to 20		More Than 20	
LOADING CONDITIONS	TYPE OF MATERIAL	Joint to Joint	Joint to Corner	Joint to Joint	Joint to Corner
Load-bearing	Clay units	140	70	100	50
	Concrete units	60	30	40	20
Nonload-bearing	Clay units	100	50	60	40
	Concrete units	50	25	30	20

For SI: 1 foot = 304.8 mm.

a. Jointing required is a minimum and is not intended to prevent minor cracking. The distances given for maximum spacing of joints are for a single-wall plane. For composite walls, the maximum spacing of joints shall be governed by the masonry material type used in the exterior wythe.

Note: To accomplish the intended purpose, joints should be located at critical locations, such as changes in building heights, changes in framing systems, columns built into exterior walls, major wall openings, and changes in materials.

2110.2 Units. Hollow or solid glass block units shall be standard or thin units.

2110.2.1 Standard units. The specified thickness of standard units shall be $3^7/_8$ inches (98 mm) thick.

2110.2.2 Thin units. The specified thickness of thin units shall be $3^1/_8$ inches (79 mm) for hollow units or 3 inches (76 mm) for solid units.

2110.3 Panel size.

2110.3.1 Exterior standard-unit panels. The maximum area of each individual exterior standard-unit panel shall be 144 square feet (13.4 m^2) when the design wind pressure is 20 psf (958 N/m^2). The maximum panel dimension between structural supports shall be 25 feet (7620 mm) in width or 20 feet (6096 mm) in height. The panel areas are permitted to be adjusted in accordance with Figure 2110.3.1 for other wind pressures.

2110.3.2 Exterior thin-unit panels. The maximum area of each individual exterior thin-unit panel shall be 85 square feet (7.9 m^2). The maximum dimension between structural supports shall be 15 feet (4572 mm) in width or 10 feet (3048 mm) in height. Thin units shall not be used in applications where the design wind pressure exceeds 20 psf (958 N/m^2).

2110.3.3 Interior panels. The maximum area of each individual standard-unit panel shall be 250 square feet (23.2 m^2). The maximum area of each thin-unit panel shall be 150 square feet (13.9 m^2). The maximum dimension between structural supports shall be 25 feet (7620 mm) in width or 20 feet (6096 mm) in height.

2110.3.4 Solid units. The maximum area of solid glass-block wall panels in both exterior and interior walls shall not be more than 100 square feet (9.3 m^2).

2110.3.5 Curved panels. The width of curved panels shall conform to the requirements of Sections 2110.3.1, 2110.3.2 and 2110.3.3, except additional structural supports shall be provided at locations where a curved section joins a straight section, and at inflection points in multicurved walls.

2110.4 Support.

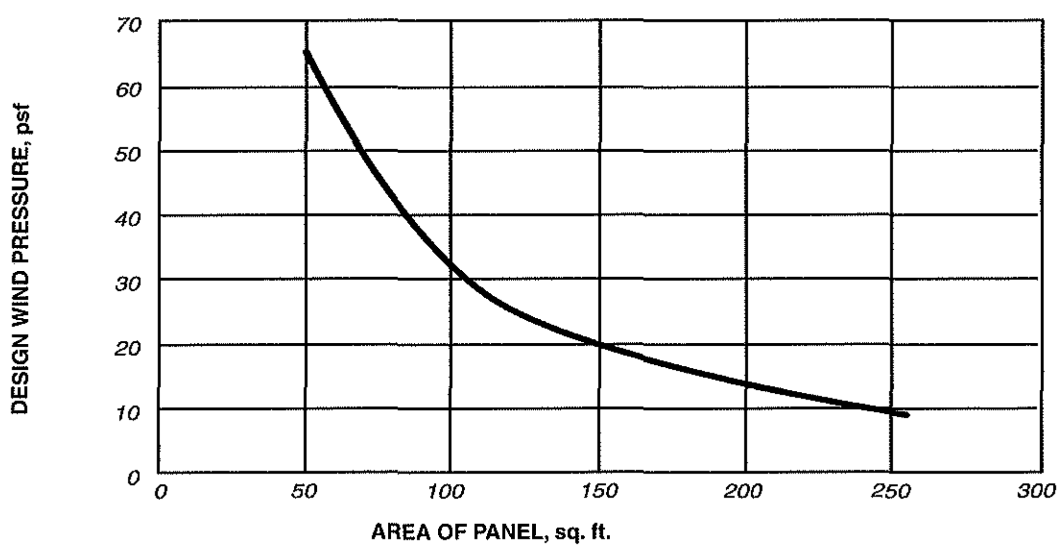
2110.4.1 Isolation. Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.

2110.4.2 Vertical. Maximum total deflection of structural members supporting glass unit masonry shall not exceed $l/600$.

2110.4.3 Lateral. Glass unit masonry panels more than one unit wide or one unit high shall be laterally supported along their tops and sides. Lateral support shall be provided by panel anchors along the top and sides spaced not more than 16 inches (406 mm) on center or by channel-type restraints. Glass unit masonry panels shall be recessed at least 1 inch (25 mm) within channels and chases. Channel-type restraints shall be oversized to accommodate expansion material in the opening, and packing and sealant between the framing restraints and the glass unit masonry perimeter units. Lateral supports for glass unit masonry panels shall be designed to resist applied loads, or a minimum of 200 pounds per lineal feet (2919 N/m) of panel, whichever is greater.

Exceptions:

1. Lateral support at the top of glass unit masonry panels that are no more than one unit wide shall not be required.



For SI: 1 square foot = 0.0929 m^2, 1 pound per square foot = 47.g N/m^2.

FIGURE 2110.3.1
GLASS MASONRY DESIGN WIND LOAD RESISTANCE

2. Lateral support at the sides of glass unit masonry panels that are no more than one unit high shall not be required.

2110.4.3.1 Single unit panels. Single unit glass unit masonry panels shall conform to the requirements of Section 2110.4.3, except lateral support shall not be provided by panel anchors.

2110.5 Expansion joints. Glass unit masonry panels shall be provided with expansion joints along the top and sides at all structural supports. Expansion joints shall have sufficient thickness to accommodate displacements of the supporting structure, but shall not be less than $^3/_8$ inch (9.5 mm) in thickness. Expansion joints shall be entirely free of mortar or other debris and shall be filled with resilient material. The sills of glass-block panels shall be coated with approved water-based asphaltic emulsion, or other elastic waterproofing material, prior to laying the first mortar course.

2110.6 Mortar. Mortar for glass unit masonry shall comply with Section 2103.7.

2110.7 Reinforcement. Glass unit masonry panels shall have horizontal joint reinforcement spaced not more than 16 inches (406 mm) on center, located in the mortar bed joint, and extending the entire length of the panel but not across expansion joints. Longitudinal wires shall be lapped a minimum of 6 inches (152 mm) at splices. Joint reinforcement shall be placed in the bed joint immediately below and above openings in the panel. The reinforcement shall have not less than two parallel longitudinal wires of size W1.7, and have welded cross wires of size W1.7.

SECTION 2111
MASONRY FIREPLACES

2111.1 General. A masonry fireplace is a fireplace constructed of concrete or masonry, hereinafter referred to as masonry. Masonry fireplaces shall be constructed in accordance with this section, Table 2111.1 and Figure 2111.1.

2111.2 Footings and foundations. Foundations for masonry fireplaces and their chimneys shall be constructed of concrete or solid masonry at least 12 inches (305 mm) thick, and shall extend at least 6 inches (152 mm) beyond the face of the fireplace or support wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 12 inches (305 mm) below finished grade.

2111.3 Seismic reinforcing. Masonry or concrete fireplaces shall be constructed, anchored, supported and reinforced as required in this chapter. In Seismic Design Category D, masonry and concrete fireplaces shall be reinforced and anchored as detailed in Sections 2111.3.1, 2111.3.2, 2111.4 and 2111.4.1 for chimneys serving fireplaces. In Seismic Design Category A, B or C, reinforcement and seismic anchorage is not required. In Seismic Design Category E or F, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2109.

2111.3.1 Vertical reinforcing. For fireplaces with chimneys up to 40 inches (1016 mm) wide, four No. 4 continuous vertical bars, anchored in the foundation, shall be placed in the concrete, between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 2103.10. For fireplaces with chimneys greater than 40 inches (1016 mm) wide, two additional No. 4 vertical bars shall be provided for each additional 40 inches (1016 mm) in width or fraction thereof.

2111.3.2 Horizontal reinforcing. Vertical reinforcement shall be placed enclosed within $^1/_4$-inch (6.4 mm) ties or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 18 inches (457 mm) on center in concrete; or placed in the bed joints of unit masonry at a minimum of every 18 inches (457 mm) of vertical height. Two such ties shall be provided at each bend in the vertical bars.

2111.4 Seismic anchorage. Masonry and concrete chimneys in Seismic Design Category D shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

2111.4.1 Anchorage. Two $^3/_{16}$-inch by 1-inch (4.8 mm by 25.4 mm) straps shall be embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two $^1/_2$-inch (12.7 mm) bolts.

2111.5 Fireplace walls. Masonry fireplaces shall be constructed of solid masonry units, hollow masonry units grouted solid, stone, or concrete. When a lining of firebrick at least 2 inches (51 mm) in thickness or other approved lining is provided, the total minimum thickness of back and side walls shall be 8 inches (203 mm) of solid masonry, including the lining. The width of joints between firebricks shall not be greater than $^1/_4$ inch (6.4 mm). When no lining is provided, the total minimum thickness of back and side walls shall be 10 inches (254 mm) of solid masonry. Firebrick shall conform to ASTM C 27 or C 1261 and shall be laid with medium-duty refractory mortar conforming to ASTM C 199.

2111.6 Steel fireplace units. Steel fireplace units incorporating a firebox liner of not less than $^1/_4$ inch (6.4 mm) in thickness and an air chamber are permitted to be installed with masonry to provide a total thickness at the back and sides of not less than 8 inches (203 mm), of which not less than 4 inches (102 mm) shall be of solid masonry. Warm-air ducts employed with steel fireplace units of the circulating air type shall be constructed of metal or masonry.

2111.7 Lintel and throat. Masonry over a fireplace opening shall be supported by a lintel of noncombustible material. The minimum required bearing length on each end of the fireplace opening shall be 4 inches (102 mm). The fireplace throat or damper shall be located a minimum of 8 inches (203 mm) above the lintel.

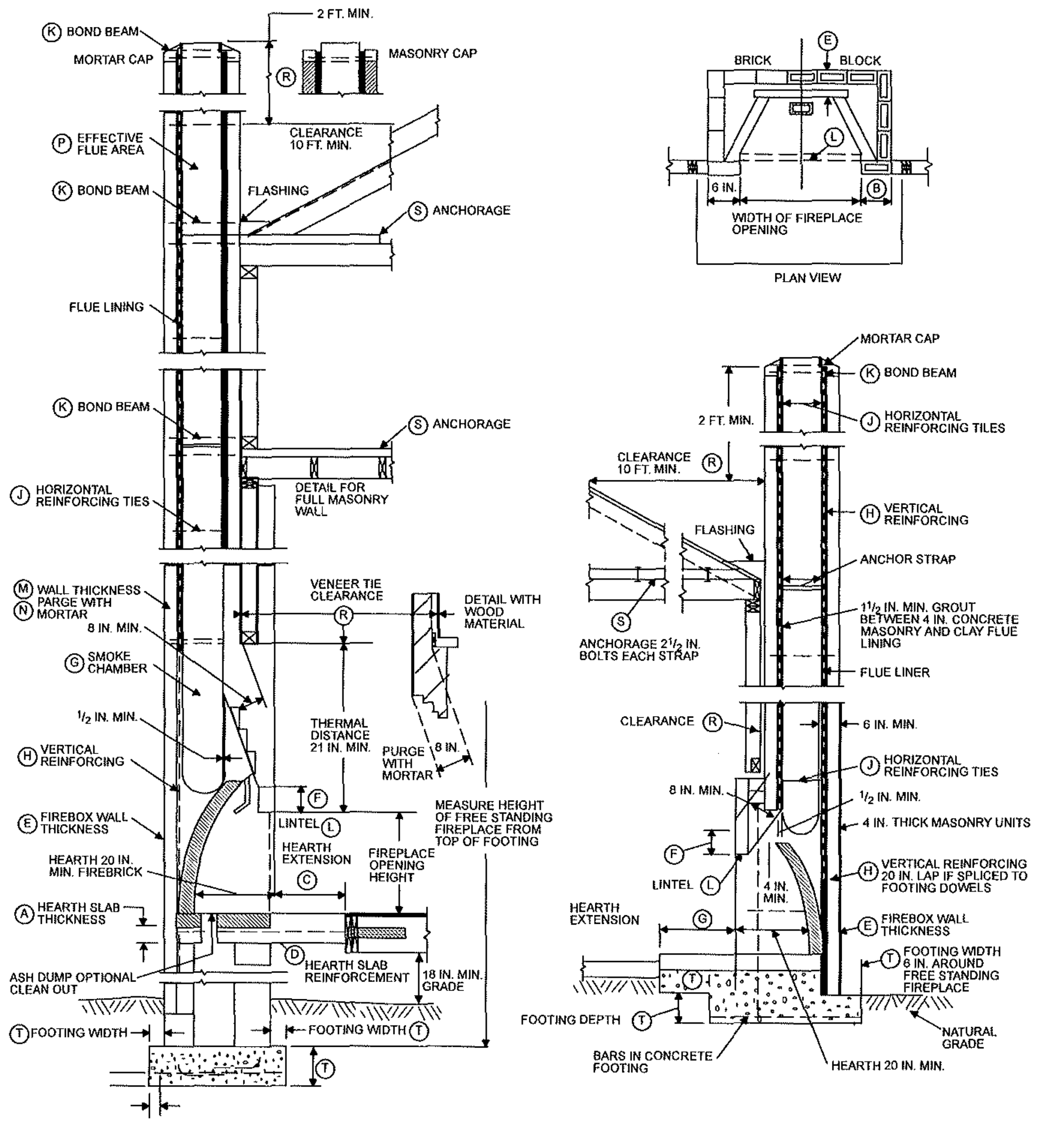
2111.8 Smoke chamber. Smoke chamber walls shall be constructed of solid masonry units, stone or concrete. Corbeling of masonry units shall not leave unit cores exposed to the inside of the smoke chamber. When a lining of firebrick at least 2 inches (51 mm) thick, or a lining of vitrified clay at least $^5/_8$ inch (16 mm) thick, is provided, the total minimum thickness of front, back and side walls shall be 6 inches (152 mm) of solid masonry, including the lining. Firebrick shall conform to ASTM C 27 or

TABLE 2111.1
SUMMARY OF REQUIREMENTS FOR MASONRY FIREPLACES AND CHIMNEYS[a]

ITEM	LETTER	REQUIREMENTS	SECTION
Hearth and hearth extension thickness	A	4-inch minimum thickness for hearth, 2-inch minimum thickness for hearth extension.	2111.9
Hearth extension (each side of opening)	B	8 inches for fireplace opening less than 6 square feet. 12 inches for fireplace opening greater than or equal to 6 square feet.	2111.10
Hearth extension (front of opening)	C	16 inches for fireplace opening less than 6 square feet. 20 inches for fireplace opening greater than or equal to 6 square feet.	2111.10
Firebox dimensions	D	20-inch minimum firebox depth. 12-inch minimum firebox depth for Rumford fireplaces.	2111.11
Hearth and hearth extension reinforcing	D	Reinforced to carry its own weight and all imposed loads.	2111.9
Thickness of wall of firebox	E	10 inches solid masonry or 8 inches where firebrick lining is used.	2111.5
Distance from top of opening to throat	F	8 inches minimum.	2111.7
Smoke chamber wall thickness dimensions	G	6 inches lined; 8 inches unlined. Not taller than opening width; walls not inclined more than 45 degrees from vertical for prefabricated smoke chamber linings or 30 degrees from vertical for corbeled masonry.	2111.8
Chimney vertical reinforcing	H	Four No. 4 full-length bars for chimney up to 40 inches wide. Add two No. 4 bars for each additional 40 inches or fraction of width, or for each additional flue.	2111.3.1, 2113.3.1
Chimney horizontal reinforcing	J	$^{1}/_{4}$-inch ties at each 18 inches, and two ties at each bend in vertical steel.	2111.3.2, 2113.3.2
Fireplace lintel	L	Noncombustible material with 4-inch bearing length of each side of opening.	2111.7
Chimney walls with flue lining	M	4-inch-thick solid masonry with $^{5}/_{8}$-inch fireclay liner or equivalent. $^{1}/_{2}$-inch grout or airspace between fireclay liner and wall.	2113.10, 2113.11, 2113.12
Effective flue area (based on area of fireplace opening and chimney)	P	See Section 2113.16.	2113.16
Clearances	R		
From chimney		2 inches interior, 1 inch exterior	2113.19
From fireplace		2 inches back or sides	2111.12
Combustible trim or materials		6 inches from opening	2111.13
Above roof		3 feet above roof penetration, 2 feet above part of structure within 10 feet.	2113.9
Anchorage	S		
Strap		$^{3}/_{16}$ inch by 1 inch	2111.4
Number		Two	2113.4.1
Embedment into chimney		12 inches hooked around outer bar with 6-inch extension.	
Fasten to		4 joists	
Bolts		Two $^{1}/_{2}$-inch diameter.	
Footing	T		
Thickness		12-inch minimum.	2111.2
Width		6 inches each side of fireplace wall.	

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2.

a. This table provides a summary of major requirements for the construction of masonry chimneys and fireplaces. Letter references are to Figure 2111.1, which shows examples of typical construction. This table does not cover all requirements, nor does it cover all aspects of the indicated requirements. For the actual mandatory requirements of the code, see the indicated section of text.



BRICK FIREBOX AND CHIMNEY—SECTIONAL SIDE VIEW ON WOOD FLOOR

BRICK FIREBOX AND BLOCK CHIMNEY—SECTIONAL SIDE VIEW ON CONCRETE SLAB

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

FIGURE 2111.1
FIREPLACE AND CHIMNEY DETAILS

C 1261 and shall be laid with refractory mortar conforming to ASTM C 199. Where no lining is provided, the total minimum thickness of front, back and side walls shall be 8 inches (203 mm) of solid masonry. When the inside surface of the smoke chamber is formed by corbeled masonry the inside surface shall be parged smooth.

2111.8.1 Smoke chamber dimensions. The inside height of the smoke chamber from the fireplace throat to the beginning of the flue shall not be greater than the inside width of the fireplace opening. The inside surface of the smoke chamber shall not be inclined more than 45 degrees from vertical when prefabricated smoke chamber linings are used or when the smoke chamber walls are rolled or sloped rather than corbeled. When the inside surface of the smoke chamber is formed by corbeled masonry, the walls shall not be corbeled more than 30 degrees from vertical.

2111.9 Hearth and hearth extension. Masonry fireplace hearths and hearth extensions shall be constructed of concrete or masonry, supported by noncombustible materials, and reinforced to carry their own weight and all imposed loads. No combustible material shall remain against the underside of hearths or hearth extensions after construction.

2111.9.1 Hearth thickness. The minimum thickness of fireplace hearths shall be 4 inches (102 mm).

2111.9.2 Hearth extension thickness. The minimum thickness of hearth extensions shall be 2 inches (51 mm).

Exception: When the bottom of the firebox opening is raised at least 8 inches (203 mm) above the top of the hearth extension, a hearth extension of not less than $^3/_8$-inch-thick (9.5 mm) brick, concrete, stone, tile or other approved noncombustible material is permitted.

2111.10 Hearth extension dimensions. Hearth extensions shall extend at least 16 inches (406 mm) in front of, and at least 8 inches (203 mm) beyond, each side of the fireplace opening. Where the fireplace opening is 6 square feet (0.557 M^2) or larger, the hearth extension shall extend at least 20 inches (508 mm) in front of, and at least 12 inches (305 mm) beyond, each side of the fireplace opening.

2111.11 Firebox dimensions. The firebox of a concrete or masonry fireplace shall have a minimum depth of 20 inches (508 mm). The throat shall not be less than 8 inches (203 mm) above the fireplace opening. The throat opening shall not be less than 4 inches (102 mm) in depth. The cross-sectional area of the passageway above the firebox, including the throat, damper and smoke chamber, shall not be less than the cross-sectional area of the flue.

Exception: Mumford fireplaces are permitted, provided that the depth of the fireplace be at least 12 inches (305 mm) and at least one-third of the width of the fireplace opening, and that the throat be at least 12 inches (305 mm) above the lintel and be at least $^1/_{20}$ the cross-sectional area of the fireplace opening.

2111.12 Fireplace clearance. Any portion of a masonry fireplace located in the interior of the building or within the exterior wall of the building shall have a minimum air space clearance to combustibles of 2 inches (51 mm). Fireplaces located entirely outside the exterior walls of the building shall have a minimum air space clearance of 1 inch (25 mm). The air space shall not be filled, except to provide fire blocking in accordance with Section 2111.14.

2111.13 Mantel and trim. Woodwork or other combustible materials shall not be placed within 6 inches (152 mm) of a fireplace opening. Combustible material within 12 inches (305 mm) of the fireplace opening shall not project more than $^1/_8$ inch (3.2 mm) for each 1-inch (25 mm) distance from such opening.

2111.14 Fireplace fireblocking. All spaces between fireplaces and floors and ceilings through which fireplaces pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 1 inch (25 mm) and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

2111.15 Ash dump cleanout. Cleanout openings, when provided, shall be equipped with ferrous metal doors and frames constructed to remain tightly closed, except when in use. Cleanouts shall be accessible and located so that ash removal will not create a hazard to combustible materials.

2111.16 Exterior air. Factory-built or masonry fireplaces covered in this section shall be equipped with an exterior air supply to ensure proper fuel combustion unless the room is mechanically ventilated and controlled so that the indoor pressure is neutral or positive.

2111.16.1 Factory-built fireplaces. Exterior combustion air ducts for factory-built fireplaces shall be a listed component of the fireplace and shall be installed according to the fireplace manufacturer's instructions.

2111.16.2 Masonry fireplaces. Listed combustion air ducts for masonry fireplaces shall be installed according to the terms of their listing and manufacturer's instructions.

2111.16.3 Exterior air intake. The exterior air intake shall be capable of providing all combustion air from the exterior of the dwelling. The exterior air intake shall not be located within the garage, attic, basement or crawl space of the dwelling nor shall the air intake be located at an elevation higher than the firebox. The exterior air intake shall be covered with a corrosion-resistant screen of $^1/_4$-inch (6.4 mm) mesh.

2111.16.4 Clearance. Unlisted combustion air ducts shall be installed with a minimum 1-inch (25 mm) clearance to combustibles for all parts of the duct within 5 feet (1524 mm) of the duct outlet.

2111.16.5 Passageway. The combustion air passageway shall be a minimum of 6 square inches (3870 mm^2) and not more than 55 square inches (0.035 m^2), except that combustion air systems for listed fireplaces or for fireplaces tested for emissions shall be constructed according to the fireplace manufacturer's instructions.

2111.16.6 Outlet. The exterior air outlet is permitted to be located in the back or sides of the firebox chamber or within 24 inches (610 mm) of the firebox opening on or near the floor. The outlet shall be closable and designed to prevent burning material from dropping into concealed combustible spaces.

SECTION 2112 MASONRY HEATERS

2112.1 Definition. A masonry heater is a heating appliance constructed of concrete or solid masonry, hereinafter referred to as masonry, having a mass of at least 1,760 pounds (800 kg) excluding the chimney and foundation, which is designed to absorb and store heat from a solid fuel fire built in the firebox by routing the exhaust gases through internal heat exchange channels in which the flow path downstream of the firebox includes at least one 180 degree (3.14 rad) change in flow direction before entering the chimney, and that delivers heat by radiation from the masonry surface of the heater that shall not exceed 230°F (110°C) except within 8 inches (203 mm) surrounding the fuel loading door(s).

2112.2 Installation. Masonry heaters shall be listed or installed in accordance with ASTM E 1602.

2112.3 Seismic reinforcing. Seismic reinforcing shall not be required within the body of a masonry heater whose height is equal to or less than 2.5 times its body width and where the masonry chimney serving the heater is not supported by the body of the heater. Where the masonry chimney shares a common wall with the facing of the masonry heater, the chimney portion of the structure shall be reinforced in accordance with Sections 2113 and 2113.4.

2112.4 Masonry heater clearance. Wood or other combustible framing shall not be placed within 4 inches (102 mm) of the outside surface of a masonry heater, provided the wall thickness of the firebox is not less than 8 inches (203 mm) and the wall thickness of the heat exchange channels is not less than 5 inches (127 mm). A clearance of at least 8 inches (203 mm) shall be provided between the gas-tight capping slab of the heater and a combustible ceiling. The required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

SECTION 2113 MASONRY CHIMNEYS

2113.1 General. A masonry chimney is a chimney constructed of concrete or masonry, hereinafter referred to as masonry. Masonry chimneys shall be constructed, anchored, supported and reinforced as required in this chapter.

2113.2 Footings and foundations. Foundations for masonry chimneys shall be constructed of concrete or solid masonry at least 12 inches (305 mm) thick and shall extend at least 6 inches (152 mm) beyond the face of the foundation or support wall on all sides. Footings shall be founded on natural undisturbed earth or engineered fill below frost depth. In areas not subjected to freezing, footings shall be at least 12 inches (305 mm) below finished grade.

2113.3 Seismic reinforcing. Masonry or concrete chimneys shall be constructed, anchored, supported and reinforced as required in this chapter. In Seismic Design Category D, masonry and concrete chimneys shall be reinforced and anchored as detailed in Sections 2113.3.1 and 2113.3.2. In Seismic Design Category A, B or C, reinforcement and seismic anchorage is not required. In Seismic Design Category E or F, masonry and concrete chimneys shall be reinforced in accordance with the requirements of Sections 2101 through 2108.

2113.3.1 Vertical reinforcing. For chimneys up to 40 inches (1016 mm) wide, four No. 4 continuous vertical bars anchored in the foundation shall be placed in the concrete, between wythes of solid masonry or within the cells of hollow unit masonry and grouted in accordance with Section 2103.10. Grout shall be prevented from bonding with the flue liner so that the flue liner is free to move with thermal expansion. For chimneys greater than 40 inches (1016 mm) wide, two additional No. 4 vertical bars shall be provided for each additional 40 inches (1016 mm) in width or fraction thereof.

2113.3.2 Horizontal reinforcing. Vertical reinforcement shall be placed enclosed within $^1/_4$-inch (6.4 mm) ties, or other reinforcing of equivalent net cross-sectional area, spaced not to exceed 18 inches (457 mm) on center in concrete, or placed in the bed joints of unit masonry, at a minimum of every 18 inches (457 mm) of vertical height. Two such ties shall be provided at each bend in the vertical bars.

2113.4 Seismic anchorage. Masonry and concrete chimneys and foundations in Seismic Design Category D shall be anchored at each floor, ceiling or roof line more than 6 feet (1829 mm) above grade, except where constructed completely within the exterior walls. Anchorage shall conform to the following requirements.

2113.4.1 Anchorage. Two $^3/_{16}$-inch by 1-inch (4.8 mm by 25.4 mm) straps shall be embedded a minimum of 12 inches (305 mm) into the chimney. Straps shall be hooked around the outer bars and extend 6 inches (152 mm) beyond the bend. Each strap shall be fastened to a minimum of four floor joists with two $^1/_2$-inch (12.7 mm) bolts.

2113.5 Corbeling. Masonry chimneys shall not be corbeled more than half of the chimney's wall thickness from a wall or foundation, nor shall a chimney be corbeled from a wall or foundation that is less than 12 inches (305 mm) in thickness unless it projects equally on each side of the wall, except that on the second story of a two-story dwelling, corbeling of chimneys on the exterior of the enclosing walls is permitted to equal the wall thickness. The projection of a single course shall not exceed one-half the unit height or one-third of the unit bed depth, whichever is less.

2113.6 Changes in dimension. The chimney wall or chimney flue lining shall not change in size or shape within 6 inches (152 mm) above or below where the chimney passes through floor components, ceiling components or roof components.

2113.7 Offsets. Where a masonry chimney is constructed with a fireclay flue liner surrounded by one wythe of masonry, the maximum offset shall be such that the centerline of the flue above the offset does not extend beyond the center of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in an approved manner, the maximum offset limitations shall not apply. Each individual corbeled masonry course of the offset shall not exceed the projection limitations specified in Section 2113.5.

2113.8 Additional load. Chimneys shall not support loads other than their own weight unless they are designed and constructed to support the additional load. Masonry chimneys are permitted to be constructed as part of the masonry walls or concrete walls of the building.

2113.9 Termination. Chimneys shall extend at least 2 feet (610 mm) higher than any portion of the building within 10 feet (3048 mm), but shall not be less than 3 feet (914 mm) above the point where the chimney passes through the roof.

2113.10 Wall thickness. Masonry chimney walls shall be constructed of concrete, solid masonry units, or hollow masonry units grouted solid with not less than 4 inches (102 mm) nominal thickness.

2113.11 Flue lining (material). Masonry chimneys shall be lined. The lining material shall be appropriate for the type of appliance connected, according to the terms of the appliance listing and manufacturer's instructions.

2113.11.1 Residential-type appliances (general). Flue lining systems shall comply with one of the following:

1. Clay flue lining complying with the requirements of ASTM C 315, Specifications for Clay Flue Linings, or equivalent.
2. Listed chimney lining systems complying with UL 1777, Chimney Liners.
3. Factory-built chimneys or chimney units listed for installation within masonry chimneys.

2113.11.1.1 Flue linings for specific appliances. Flue linings other than those covered in Section 2113.11.1 intended for use with specific appliances shall comply with Sections 2113.11.1.2 through 2113.11.1.4 and Sections 2113.11.2 and 2113.11.3.

2113.11.1.2 Gas appliances. Flue lining systems for gas appliances shall be in accordance with the *International Fuel Gas Code.*

2113.11.1.3 Pellet fuel-burning appliances. Flue lining and vent systems for use in masonry chimneys with pellet fuel-burning appliances shall be limited to flue lining systems complying with Section 2113.11.1 and pellet vents listed for installation within masonry chimneys. (See Section 2113.11.1.5 for marking.)

2113.11.1.4 Oil-fired appliances approved for use with L-vent. Flue lining and vent systems for use in masonry chimneys with oil-fired appliances approved for use with Type L vent shall be limited to flue lining systems complying with Section 2113.11.1 and listed chimney liners complying with UL 641. (See Section 2113.11.1.5 for marking.)

2113.11.1.5 Notice of usage. When a flue is relined with a material not complying with Section 2113.11.1, the chimney shall be plainly and permanently identified by a label attached to a wall, ceiling or other conspicuous location adjacent to where the connector enters the chimney. The label shall include the following message or equivalent language: "This chimney is for use only with (Type or category of appliance) that burns (type of fuel). Do not connect other types of appliances."

2113.11.2 Concrete and masonry chimneys for medium heat appliances.

2113.11.2.1 General. Concrete and masonry chimneys for medium-heat appliances shall comply with Sections 2113.1 through 2113.5.

2113.11.2.2 Construction. Chimneys for medium-heat appliances shall be constructed of solid masonry units or of concrete with walls a minimum of 8 inches (203 mm) thick, or with stone masonry a minimum of 12 inches (305 mm) thick.

2113.11.2.3 Lining. Concrete and masonry chimneys shall be lined with an approved medium-duty refractory brick a minimum of $4^1/_2$ inches (114 mm) thick laid on the $4^1/_2$-inch bed (114 mm) in an approved medium-duty refractory mortar. The lining shall start 2 feet (610 mm) or more below the lowest chimney connector entrance. Chimneys terminating 25 feet (7620 mm) or less above a chimney connector entrance shall be lined to the top.

2113.11.2.4 Multiple passageway. Concrete and masonry chimneys containing more than one passageway shall have the liners separated by a minimum 4-inch-thick (102 mm) concrete or solid masonry wall.

2113.11.2.5 Termination height. Concrete and masonry chimneys for medium-heat appliances shall extend a minimum of 10 feet (3048 mm) higher than any portion of any building within 25 feet (7620 mm).

2113.11.2.6 Clearance. A minimum clearance of 4 inches (102 mm) shall be provided between the exterior surfaces of a concrete or masonry chimney for medium-heat appliances and combustible material.

2113.11.3 Concrete and masonry chimneys for high-heat appliances.

2113.11.3.1 General. Concrete and masonry chimneys for high-heat appliances shall comply with Sections 2113.1 through 2113.5.

2113.11.3.2 Construction. Chimneys for high-heat appliances shall be constructed with double walls of solid masonry units or of concrete, each wall to be a minimum of 8 inches (203 mm) thick with a minimum air space of 2 inches (51 mm) between the walls.

2113.11.3.3 Lining. The inside of the interior wall shall be lined with an approved high-duty refractory brick, a minimum of $4^1/_2$ inches (114 mm) thick laid on the $4^1/_2$-inch bed (114 mm) in an approved high-duty refractory mortar. The lining shall start at the base of the chimney and extend continuously to the top.

2113.11.3.4 Termination height. Concrete and masonry chimneys for high-heat appliances shall extend a minimum of 20 feet (6096 mm) higher than any portion of any building within 50 feet (15240 mm).

2113.11.3.5 Clearance. Concrete and masonry chimneys for high-heat appliances shall have approved clearance from buildings and structures to prevent overheating combustible materials, permit inspection and maintenance operations on the chimney, and prevent danger of burns to persons.

2113.12 Flue lining (installation). Flue liners shall be installed in accordance with ASTM C 1283 and extend from a point not less than 8 inches (203 mm) below the lowest inlet or, in the case of fireplaces, from the top of the smoke chamber, to a point above the enclosing walls. The lining shall be carried up vertically, with a maximum slope no greater than 30 degrees from the vertical.

Fireclay flue liners shall be laid in medium-duty refractory mortar conforming to ASTM C 199, with tight mortar joints left smooth on the inside and installed to maintain an air space or insulation not to exceed the thickness of the flue liner separating the flue liners from the interior face of the chimney masonry walls. Flue lining shall be supported on all sides. Only enough mortar shall be placed to make the joint and hold the liners in position.

2113.13 Additional requirements.

2113.13.1 Listed materials. Listed materials used as flue linings shall be installed in accordance with the terms of their listings and manufacturer's instructions.

2113.13.2 Space around lining. The space surrounding a chimney lining system or vent installed within a masonry chimney shall not be used to vent any other appliance.

Exception: This shall not prevent the installation of a separate flue lining in accordance with the manufacturer's installation instructions.

2113.14 Multiple flues. When two or more flues are located in the same chimney, masonry wythes shall be built between adjacent flue linings. The masonry wythes shall be at least 4 inches (102 mm) thick and bonded into the walls of the chimney.

Exception: When venting only one appliance, two flues are permitted to adjoin each other in the same chimney with only the flue lining separation between them. The joints of the adjacent flue linings shall be staggered at least 4 inches (102 mm).

2113.15 Flue area (appliance). Chimney flues shall not be smaller in area than that of the area of the connector from the appliance. The sizing of a chimney flue to which multiple-appliance venting systems are connected shall be in accordance with Section M1805.3 or Chapter 24 of the *International Residential Code.*

2113.16 Flue area (masonry fireplace). Flue sizing for chimneys serving fireplaces shall be in accordance with Section 2113.16.1 or Section 2113.16.2.

2113.16.1 Minimum area. Round chimney flues shall have a minimum net cross-sectional area of at least $^1/_{12}$ of the fireplace opening. Square chimney flues shall have a minimum net cross-sectional area of at least $^1/_{10}$ of the fireplace opening. Rectangular chimney flues with an aspect ratio less than 2 to 1 shall have a minimum net cross-sectional area of at least $^1/_{10}$ of the fireplace opening. Rectangular chimney flues with an aspect ratio of 2 to 1 or more shall have a minimum net cross-sectional area of at least $^1/_8$ of the fireplace opening.

2113.16.2 Determination of minimum area. The minimum net cross-sectional area of the flue shall be determined in accordance with Figure 2113.16. A flue size providing at least the equivalent net cross-sectional area shall be used. Cross-sectional areas of clay flue linings are provided in Tables 2113.16(1) and 2113.16(2) or as provided by the manufacturer or as measured in the field. The height of the chimney shall be measured from the firebox floor to the top of the chimney flue.

TABLE 2113.16(1)
NET CROSS-SECTIONAL AREA OF ROUND FLUE SIZES[a]

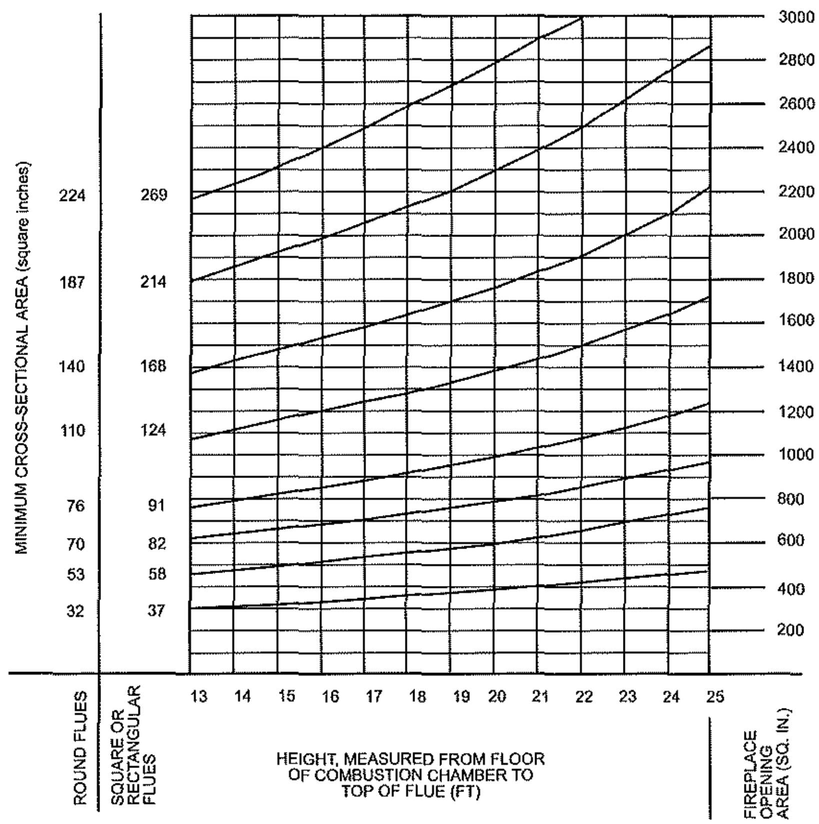
FLUE SIZE, INSIDE DIAMETER (inches)	CROSS-SECTIONAL AREA (square inches)
6	28
7	38
8	50
10	78
$10\,^3/_4$	90
12	113
15	176
18	254

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2.
a. Flue sizes are based on ASTM C 315.

TABLE 2113.16(2)
NET CROSS-SECTIONAL AREA OF SQUARE AND RECTANGULAR FLUE SIZES[a]

FLUE SIZE, INSIDE DIMENSION (inches)	CROSS-SECTIONAL AREA (square inches)
$4^1/_2 \times 13$	34
$7^1/_2 \times 7^1/_2$	37
$8^1/_2 \times 8^1/_2$	47
$7^1/_2 \times 11^1/_2$	58
$8^1/_2 \times 13$	74
$7^1/_2 \times 15^1/_2$	82
$11^1/_2 \times 11^1/_2$	91
$8^1/_2 \times 17^1/_2$	101
13×13	122
$11^1/_2 \times 15^1/_2$	124
$13 \times 17^1/_2$	165
$15^1/_2 \times 15^1/_2$	168
$15^1/_2 \times 19^1/_2$	214
$17^1/_2 \times 17^1/_2$	226
$19^1/_2 \times 19^1/_2$	269
20×20	286

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2.
a. Flue sizes are based on ASTM C 315.



For SI: 1 inch = 25.4 mm, 1 square inch = 645 mm^2.

FIGURE 2113.16
FLUE SIZES FOR MASONRY CHIMNEYS

2113.17 Inlet. Inlets to masonry chimneys shall enter from the side. Inlets shall have a thimble of fireclay, rigid refractory material or metal that will prevent the connector from pulling out of the inlet or from extending beyond the wall of the liner.

2113.18 Masonry chimney cleanout openings. Cleanout openings shall be provided within 6 inches (152 mm) of the base of each flue within every masonry chimney. The upper edge of the cleanout shall be located at least 6 inches (152 mm) below the lowest chimney inlet opening. The height of the opening shall be at least 6 inches (152 mm). The cleanout shall be provided with a noncombustible cover.

Exception: Chimney flues serving masonry fireplaces, where cleaning is possible through the fireplace opening.

2113.19 Chimney clearances. Any portion of a masonry chimney located in the interior of the building or within the exterior wall of the building shall have a minimum air space clearance to combustibles of 2 inches (51 mm). Chimneys located entirely outside the exterior walls of the building, including chimneys that pass through the soffit or cornice, shall have a minimum air space clearance of 1 inch (25 mm). The air space shall not be filled, except to provide fireblocking in accordance with Section 2113.20.

Exception: Masonry chimneys equipped with a chimney lining system listed and labeled for use in chimneys in contact with combustibles in accordance with UL 1777 and installed in accordance with the manufacturer's installation instructions are permitted to have combustible material in contact with their exterior surfaces. However, this shall not eliminate the requirement for noncombustible fireblocking in accordance with Section 2113.20.

2113.20 Chimney fireblocking. All spaces between chimneys and floors and ceilings through which chimneys pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be to a depth of 1 inch (25 mm) and shall only be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

CHAPTER 22

STEEL

SECTION 2201 GENERAL

2201.1 Scope. The provisions of this chapter govern the quality, design, fabrication and erection of steel used structurally in buildings or structures.

SECTION 2202 DEFINITIONS AND NOMENCLATURE

2202.1 Definitions. The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meaning shown herein.

STEEL CONSTRUCTION, COLD-FORMED. That type of construction made up entirely, or in part of steel structural members cold formed to shape from sheet or strip steel such as roof deck, floor and wall panels, studs, floor joists, roof joists and other structural elements.

STEEL JOIST. Any steel structural member of a building or structure made of hot-rolled or cold-formed solid or open-web sections, or riveted or welded bars, strip or sheet steel members, or slotted and expanded, or otherwise deformed rolled sections.

STEEL MEMBER, STRUCTURAL. Any steel structural member of a building or structure consisting of a rolled steel structural shape other than cold-formed steel, or steel joist members.

2202.2 Nomenclature. The following symbols shall, for the purposes of this chapter and as used elsewhere in this code, have the meaning shown herein.

ϕ Resistance factor (Section 2211.6).

Ω Factor of safety (Section 2211.6).

Ω_o System overstrength factor (Table 1617.6).

SECTION 2203 IDENTIFICATION AND PROTECTION OF STEEL FOR STRUCTURES

2203.1 Identification. Steel furnished for structural load- carrying purposes shall be properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this chapter. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

2203.2 Protection. Painting of structural steel shall comply with the requirements contained in either the AISC-LRFD or AISC-ASD or AISC-HSS. Individual structural members and assembled panels of cold-formed steel construction, except where fabricated of approved corrosion resistant steel or of steel having a corrosion resistant or other approved coating, shall be protected against corrosion with an approved coat of paint, enamel or other approved protection.

SECTION 2204 STRUCTURAL STEEL CONSTRUCTION

2204.1 General. The design, fabrication and erection of structural steel for buildings and structures shall be in accordance with either the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings (AISC-LRFD), AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD) or AISC Specification for the Design of Steel Hollow Structural Sections (AISC-HSS). Where required, the seismic design of steel structures shall be in accordance with the additional provisions of Section 2212.

SECTION 2205 COLD-FORMED STEEL

2205.1 General. The design of cold-formed carbon and low alloy steel structural members shall be in accordance with the AISI Specification for the Design of Cold-Formed Steel Structural Members. The design of cold-formed stainless steel structural members shall be in accordance with ASCE 8. Where required, the seismic design of cold-formed steel structural members shall be in accordance with the additional provisions of Section 2211.

2205.2 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3.

SECTION 2206 STEEL JOISTS

2206.1 General. The design, manufacturing and use of open web steel joists and joist girders shall be in accordance with one of the following SJI specifications:

1. Standard Specifications for Open Web Steel Joists, K Series.
2. Standard Specifications for Longspan Steel Joists, LH Series and Deep Longspan Steel Joists, DLH Series.
3. Standard Specifications for Joist Girders.

Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 2211.

SECTION 2207 STEEL CABLE STRUCTURES

2207.1 General. The design, fabrication, and erection including related connections, and protective coatings of steel cables for buildings shall be in accordance with ASCE 19.

2207.2 Seismic requirements for steel cable. The design strength of steel cables shall be determined by the provisions of ASCE 19 except as modified by these provisions. Section 5d of ASCE 19 shall be modified by substituting $1.5(T_4)$ where T_4 is the net tension in cable due to dead load, prestress, live load and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of ASCE 19.

SECTION 2208 WELDING

2208.1 [Comm 62.2208] Welding. The details of design, workmanship and technique for welding, inspection of welding, and qualification of welding operators shall conform to the requirements of the specifications listed in Sections 2204, 2205, 2206 and 2207.

Note: The rules pertaining to registration of structural welders are specified in ch. Comm 5.

SECTION 2209 BOLTING

2209.1 General. The design, installation and inspection of bolts shall be in accordance with the requirements of the specifications listed in Sections 2204 and 2205. Special inspection of the installation of high strength bolts shall be provided where required by Section 1704.

2209.2 Anchor bolts. Anchor bolts shall be set accurately to the pattern and dimensions called for on the plans. The protrusion of the threaded ends through the connected material shall be sufficient to fully engage the threads of the nuts, but shall not be greater than the length of the threads on the bolts.

SECTION 2210 STEEL STORAGE RACKS

2210.1 Storage racks. The design, testing and utilization of industrial steel storage racks shall be in accordance with the RMI Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks. Racks included in the scope of this specification include industrial pallet racks, movable shelf racks and stacker racks, and does not apply to other types of racks, such as drive-in and drive-through racks, cantilever racks, portable racks or rack buildings. Where required, the seismic design of storage racks shall be in accordance with the provisions of Section 1621.2.8.

SECTION 2211 WIND AND SEISMIC REQUIREMENTS FOR LIGHT-FRAMED COLD-FORMED STEEL WALLS

2211.1 General. The design of light-framed walls of cold-formed carbon or low-alloy steel to resist wind and seismic loads shall be in accordance with the provisions of AISI or ASCE 8 and the additional requirements of this section. Where shear panels, attached to light-framed cold-formed steel framing members, are used to resist lateral forces produced by wind or seismic loads, the nominal shear value used to establish the allowable shear values or design shear values are given as shown in Table 2211.1(1) or 2211.1(2) for wind loads or Table 2211.1(3) for seismic loads. The allowable shear value (ASD) or design shear value (LRFD) shall be determined using the factor of safety Ω or resistance factor ϕ as set forth in Section 2211.6.

2211.2 Conditions of application. Boundary elements and connections thereto shall be proportioned to transmit the induced forces. For connections, screws shall be of sufficient length to penetrate through the cold-formed steel framing member by at least three exposed threads. Framing screws shall be a minimum in accordance with SAE J78 and shall have a minimum edge distance of 0.5 inch (12.7 mm). Screws required to conform to SAE J78 shall have a Type II coating in accordance with ASTM B 633.

2211.2.1 Limitations for systems in Tables 2211.1(1), 2211.1(2) and 2211.1(3). The lateral-resistant systems listed in Tables 2211.1(1), 2211.1(2) and 2211.1(3) shall conform to the following requirements:

1. Studs shall be a minimum $1^5/_8$ inches (41 mm) by $3^1/_2$ inches (89 mm) with a $^3/_8$-inch (9.5 mm) return lip. As a minimum, studs shall be doubled (back to back) at shear wall ends.
2. Track shall be a minimum $1^1/_4$ inches (32 mm) by$3^1/_2$ inches (89 mm).
3. Both studs and track shall have a minimum uncoated base metal thickness of 0.033 inch (0.84 mm) and shall be of the following grades of structural quality steel: ASTM A 653 SS Grade 33, ASTM A 792 SS Grade 33 or ASTM A 875 SS Grade 33.
4. Fasteners along the edges in shear panels shall be placed not less than $^3/_8$ inch (9.5 mm) in from panel edges.
5. The height-to-length ratio of wall systems shall not exceed the values in Tables 2211.1(1), 2211.1(2) and 2211.1(3).
6. Panel thicknesses shown are minimums. Panels less than 12 inches (305 mm) wide shall not be used. Panel edges shall be fully blocked. Where horizontal strapping is used to provide such blocking, it shall be a minimum $1^1/_2$ inches (38 mm) wide and of the same material and thickness as the track and studs.

2211.3 Wood structural panel sheathing. Light-framed cold-formed steel wall systems sheathed with wood structural panels are permitted to resist horizontal forces produced by wind or seismic loads.

2211.3.1 Shear values. Nominal shear values used to establish the allowable shear value are given for design shear value in Table 2211.1(1) for wind loads and Table 2211.1(3) for seismic loads. As an alternative to the provisions in Tables 2211.1(1) and 2211.1(3), shear values are permitted to be calculated by the principles of mechanics by using wood structural panel shear values and approved fastener values. Wood structural panels shall comply with DOC PS 1 or PS 2 and shall be manufactured using exterior glue in accordance with Section 2303.1.4. Where $^7/_{16}$-inch (11.1 mm) OSB is specified, $^{15}/_{32}$-inch (11.9 mm) Structural 1 sheathing (plywood) is permitted. Increases of the nominal loads shown in Tables 2211.1(1) and 2211.1(3) shall not be permitted for duration of load or installing sheathing on both sides of a wall unless otherwise permitted herein.

TABLE 2211.1(1)
NOMINAL SHEAR VALUES FOR WIND FORCES IN POUNDS PER FOOT FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS[a]

ASSEMBLY DESCRIPTION	MAXIMUM HEIGHT/LENGTH RATIO	FASTENER SPACING AT PANEL EDGES[b] (inches)				MAXIMUM FRAMING SPACING (inches o.c.)
		6	4	3	2	
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) one side	2:1	1,065[c]	—	—	—	24
$^{7}/_{16}$-inch Rated Sheathing (OSB) one side	2:1	910[c]	1,410	1,735	1,910	24
$^{7}/_{16}$-inch Rated Sheathing (OSB) one side oriented perpendicular to framing	2:1	1,020[c]	—	—	—	24
$^{7}/_{16}$-inch Rated Sheathing (OSB) one side	4:1	—	1,025	1,425	1,825	24
0.018-inch Steel sheet, one side	2:1	485	—	—	—	24
0.027-inch Steel sheet, one side	4:1	—	1,000	—	—	24

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.

a. Nominal shear values shall be multiplied by the resistance factor ϕ to determine design strength or divided by the safety factor Ω to determine allowable shear values as set forth in Section 2211.6.

b. Screws in the field of the panel shall be installed 12 inches o.c. unless otherwise shown.

c. Where fully blocked gypsum board is applied to the opposite side of this assembly, per Table 2211.1(2) with screw spacing at 7 inches o.c. edge and 7 inches o.c. field, these nominal values are permitted to be increased by 30 percent.

TABLE 2211.1(2)
NOMINAL SHEAR VALUES FOR WIND FORCES IN POUNDS PER FOOT FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS AND FACED WITH GYPSUM BOARD[a]

WALL CONSTRUCTION	MAXIMUM HEIGHT/LENGTH RATIO	ORIENTATION	SCREW SPACING (inches)		NOMINAL SHEAR VALUE (plf)
			Edge	Field	
$^{1}/_{2}$-inch gypsum board on both sides of wall; Studs maximum 24 inch o.c.	2:1	Gypsum board applied perpendicular to framing with strap blocking behind the horizontal joint and with solid blocking between the first two end studs	7	7	585
			4	4	850

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.

a. Nominal shear values shall be multiplied by the resistance factor ϕ to determine design strength or divided by the safety factor Ω to determine allowable shear values as set forth in Section 2211.6.

2211.3.2 Orientation. Structural panels are permitted to be applied either parallel to or perpendicular to framing.

2211.3.3 Attachment. Screws used to attach plywood and OSB shall be approved and shall be a minimum No. 8 flathead self-drilling tapping screws with a minimum head diameter of 0.292 inch (7.42 mm) in accordance with SAE J78. Such screws shall be of sufficient length to penetrate through the cold-formed steel framing member by at least three exposed threads.

2211.4 Gypsum board panel sheathing. Cold-formed steel stud wall systems sheathed with gypsum board are permitted to resist horizontal forces produced by wind loads where the nominal shear value used to establish the allowable shear value or design shear value does not exceed the value set forth in Table 2211.1(2).

2211.4.1 Shear values. The shear values listed in Table 2211.1(2) shall not be cumulative with the shear values of other materials applied to the same wall unless otherwise permitted herein. The nominal shear values shown shall not be used unless gypsum board is applied to both sides of the wall in the manner shown nor shall they be proportionally reduced to obtain nominal shear values for cold-formed steel stud walls with gypsum board applied to one side only.

2211.4.2 Orientation. Gypsum board shall be applied perpendicular to studs in accordance with Table 2211.1(2). End joints of adjacent courses of gypsum board sheets shall not occur over the same stud.

2211.4.3 Attachment. Screws used to attach gypsum board shall be a minimum No. 6 in accordance with ASTM C 954, and shall be of sufficient length to penetrate into the cold-formed steel framing member by at least three exposed threads.

2211.5 Sheet steel sheathing. Light-framed cold-formed steel wall systems sheathed with steel sheets are permitted to resist horizontal forces produced by wind or seismic loads. Steel sheets shall have a minimum base metal thickness as shown in

TABLE 2211.1(3)
NOMINAL SHEAR VALUES FOR SEISMIC FORCES IN POUNDS PER FOOT FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS[a]

ASSEMBLY DESCRIPTION	MAXIMUM HEIGHT/LENGTH RATIO	FASTENER SPACING AT PANEL EDGES[b] (inches)				MAXIMUM FRAMING SPACING (inches o.c.)
		6	4	3	2	
$^{15}/_{32}$-inch Structural 1 Sheathing(4-ply) plywood one side	2:1[c]	780	990	1,465	1,625	24
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) plywood one side; endstuds 0.043-inch min. thickness	2:1	—	—	1,775	2,190	24
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) plywood one side; all studs and track) 0.043-inch min. thickness	2:1	890	1,330	1,775	2,190	24
$^{7}/_{16}$-inch OSB one side	2:1[c]	700	915	1,275	1,625	24
$^{7}/_{16}$-inch OSB one side end studs, 0.043 inch min. thickness	2:1	—	—	1,520	2,060	24
0.018-inch min. thickness steel sheet one side	2:1	390	—	—	—	24
0.027-inch min. thickness steel sheet one side	2:1[c]	—	1,000	1,085	1,170	24

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.

a. Nominal shear values shall be multiplied by the resistance factor ϕ to determine design strength or divided by the safety factor Ω to determine allowable shear values as set forth in Section 2211.6. Nominal shear values shall not be increased for material applied on both sides; see Section 2211.3.

b. Screws in the field of the panel shall be installed 12 inches o.c. unless otherwise shown.

c. In Seismic Design Categories A through C, the height to width ratio is permitted to be 4:1.

Tables 2211.1(1) or 2211.1(3) and shall be of the following grades of structural quality steel: ASTM A 653 SS Grade 33, ASTM A 792 SS Grade 33 or ASTM A 875 SS Grade 33.

2211.5.1 Shear values. Nominal shear values used to establish the allowable shear value or design shear value are given in Table 2211.1(1) for wind loads and 2211.1(3) for seismic loads. Installing sheathing on both sides of a wall to increase the shear resistance shall not be permitted unless otherwise shown.

2211.5.2 Orientation. Steel sheets are permitted to be applied either parallel to or perpendicular to framing.

2211.5.3 Attachment. Screws used to attach steel sheets shall be a minimum No. 8 modified truss head, and shall be of sufficient length to penetrate into the cold-formed steel framing member by at least three exposed threads.

2211.6 Design shear determination. Where allowable stress design is used, the allowable shear value shall be determined by dividing the nominal shear value, shown in Tables 2211.1(1), 2211.1(2) and 2211.1(3), by a factor of safety (Ω) of 2.5. Where Load and Resistance Factor Design is used, the design shear value shall be determined by multiplying the nominal shear value, shown in Tables 2211.1(1), 2211.1(2) and 2211.1(3), by a resistance factor (ϕ) of 0.55.

2211.7 Seismic Design Category D, E or F. In addition to the requirements of Sections 2205 and 2211, cold-formed steel stud wall systems in buildings assigned to Seismic Design Category D, E or F, in accordance with Section 1616, shall comply with the requirements of this section.

2211.7.1 Boundary elements. Boundary elements, chords and collectors shall be designed to transmit the induced design axial forces.

2211.7.2 Connections. Connections for diagonal bracing members, top chord splices, boundary elements and collectors shall be designed to develop the lesser of the nominal tensile strength of the member or the design seismic force multiplied by Ω_0, where Ω_0 is the system overstrength factor from Table 1617.6. The pull-out resistance of screws shall not be used to resist design seismic forces.

2211.7.3 Braced bay members. Vertical and diagonal members in braced bays shall be anchored such that the bottom track is not required to resist uplift by bending of the track web. Both flanges of the studs shall be braced to prevent lateral torsional buckling. Vertical boundary elements and anchorage thereto shall have the strength to resist the forces determined from the application of load combinations in accordance with Section 1605.4.

2211.7.4 Wood structural panel sheathing. Where wood structural panels provide lateral resistance, the design and construction of such walls shall be in accordance with the additional requirements of this section. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Wood sheathing shall not be used to splice these members. Wall studs and track shall have a minimum uncoated base metal thickness of 0.033 inch (0.838 mm) and shall not have a uncoated base metal thickness greater than 0.048 inch (1.22 mm). The nominal shear value for cold-formed steel stud wall systems for buildings in Seismic Design Categories D and E shall be based on values from Table 2211.1(3). Calculation of shear values in accordance with Section 2211.3 shall not be permitted.

2211.7.5 Diagonal bracing. Where diagonal bracing is provided for lateral resistance it shall comply with the requirements of this section. The *l/r* of the brace is permitted to exceed 200. Provisions shall be made for pretensioning or other methods of installing tension-only bracing shall be used to guard against loose diagonal straps.

SECTION 2212 SEISMIC REQUIREMENTS FOR STRUCTURAL STEEL CONSTRUCTION

2212.1 Seismic requirements for steel structures. The design of steel structures to resist seismic forces shall be in accordance with the provisions of Section 2212.1.1 or 2212.1.2 for the appropriate seismic design category.

2212.1.1 Seismic Design Category A, B or C. Steel structures assigned to Seismic Design Category A, B or C, in accordance with Section 1616, shall be of any construction permitted in Sections 2204 through 2207 inclusive. An *R* factor as set forth in Table 1617.6 for the appropriate steel system is permitted where the structure is designed and detailed in accordance with the provisions of AISC Seismic Part I or III or Section 2211, for light-framed cold-formed steel wall systems. Systems not detailed in accordance with the above shall use the *R* factor in Table 1617.6 designated for "steel systems not detailed for seismic."

2212.1.2 Seismic Design Category D, E or F. Steel structures assigned to Seismic Design Category D, E or F shall be designed and detailed in accordance with AISC Seismic Part I or III or Section 2211.

SECTION 2213 SEISMIC REQUIREMENTS FOR COMPOSITE CONSTRUCTION

2213.1 General. The design, construction, and quality of composite steel and concrete components that resist seismic forces shall conform to the requirements of the AISC LRFD and ACI 318. An *R* factor as set forth in Table 1617.6 for the appropriate composite steel and concrete system is permitted where the structure is designed and detailed in accordance with the provisions of AISC Seismic Part II. In Seismic Design Category B or above, the design of such systems shall conform to the requirements of AISC Seismic Part II.

2213.2 Seismic Design Category D, E or F. Composite structures are permitted in Seismic Design Category D or above, subject to the limitations in Table 1617.6, where substantiating evidence is provided to demonstrate that the proposed system will perform as intended by AISC Seismic Part II. The substantiating evidence shall be subject to building official approval. Where composite elements or connections are required to sustain inelastic deformations, the substantiating evidence shall be based on cyclic testing.