1997 UBC
Structural
CheckList ™

Based on the 1997 *Uniform Building Code*™ (UBC)

PREFACE

The International Conference of Building Officials is pleased to offer the following set of checklists:

1997 UBC Accessibility CheckList™

1997 UBC Nonstructural CheckList™

1997 UBC Structural CheckList™

1997 UMC CheckList™

1997 UFC CheckList™

The checklists have been developed to facilitate the architect, designer, civil engineer and plan reviewer in the plan review process. These checklists are comprehensive and, as a result, somewhat lengthy. However, they have been organized and categorized into subjects that allow the user to quickly identify which comments are applicable to the project. A code section is referenced within each comment so that the user may refer to the code provision for further clarification. The comments in the *UBC Structural CheckList* have been further classified with a subject title for each comment because of the technical nature of the list.

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The *UBC Structural CheckList* is intended to serve as a guide in completing a plan review and does not include all possible comments.

GENERAL

I. GENERAL

A. Required Plans, Reports and Structural Calculations

1. **Plans**

   a. **Licensed Architect or Engineer.** Plans, calculations and specifications should be prepared by a licensed architect or engineer licensed by the state to practice as such. (Section 106.3.2)

   b. **Deferred Submittal.** Those portions of the design which are not submitted at the time of application should be listed as deferred submittal on the plans. (Section 106.3.4.2)

   c. **Grid System.** A grid system should be provided on the plans to facilitate reference to the plans. Also, cross-referencing calculations to the grid system is encouraged to facilitate the plan review process. (Section 106.3.3)

   d. **Required Details.** Details should be provided on the plans for items designed in the calculations. (Section 106.3.3)

2. **Calculations**

   a. **Clear, Legible and Organized Calculations.** Calculations should be presented in a clear, legible and organized manner conducive to structural plan review. A table of contents should be provided and pages should be numbered for referencing purposes. (Section 106.3.3)

   b. **Key Plan.** A key plan should be provided for the structural members designed on pages _____ of the calculations so that the calculations and plans may be correlated. (Section 106.3.3)

3. **Soils Report**

   a. **Required Soils Report.** A soils investigation report should be submitted for review in accordance with Section 1804.3.

   b. **Soils Report Recommendations.** Building plans and calculations should follow the recommendations of the soils report. (Section 1804.3)

B. Explanations for Computer Programs, Input and Output

1. **User's Guide and Explanation Comments.** A program description (User's Guide) should be provided and contain information necessary to determine the following:

   a. Nature and extent of the analysis.

b.   Verification of the input data.

c.   Interpretation of results.

d.   Compliance of the computations with the code.

Also, explanation comments by the designer beside the input and output data and a written summary that provides the controlling loads from the computer output in a graphical manner are encouraged to facilitate the structural plan review. (Sections 106.3.2 and 106.3.3)

2. **Identification of Computer Input.** Data provided as computer input should be clearly distinguished from those computed in the program. (Sections 106.3.2 and 106.3.3)

3. **Correlation between Members and Plans.** The member identification used in the program input and results should be readily correlated with the members shown on the plans. (Sections 106.3.2 and 106.3.3)

C.   Special Inspection and Structural Observation Requirements

1. Plans should indicate that special inspection will be provided for the following work: (Section 1701.1)

    a. **Concrete.** During the taking of test specimens and placing of reinforced concrete. (Section 1701.5, Item 1)

    b. **Bolts Installed in Concrete.** Prior to and during the placement of concrete around bolts when stress increases permitted by Footnote 5 of Table 19-D or Section 1923 are utilized. (Section 1701.5, Item 2)

    c. **Concrete SMRFs.** During the placement of the reinforcement and concrete of special moment-resisting frames (SMRFs) in Seismic Zones 3 and 4. (Section 1701.5, Item 3)

    d. **Prestressed Concrete Steel.** During placing, stressing and grouting of tendons in prestressed concrete. (Section 1701.5, Item 4)

    e. **Structural Steel Welding.** (Section 1701.5, Item 5)

    f. **Reinforcing Steel Welding.** (Section 1701.5, Item 5.3)

    g. **High-strength Bolting.** (Section 1701.5, Item 6)

    h. **Structural Masonry.** See Section 1701.5, Item 7.1, for masonry other than fully grouted open-end hollow-unit masonry. See Section 1701.5, Item 7.2, for fully grouted open-end hollow-unit masonry.

GENERAL

     i.   **Reinforced Gypsum Concrete.** (Section 1701.5, Item 8)

     j.   **Insulating Concrete Fill.** (Section 1701.5, Item 9)

     k.   **Piling, Drilled Piers and Caissons.** During driving and testing of piles and construction of cast–in–place drilled piles or caissons. (Section 1701.5, Item 11)

     l.   **Shotcrete.** During the taking of test specimens and placement of shotcrete. (Section 1701.5, Item 12)

     m.   **Special Grading, Excavation and Filling.** (Section 1701.5, Item 13)

2. **Report by Special Inspector.** Plans should note that a final signed report will be submitted by the special inspector in accordance with Section 1701.3.

3. **Structural Observation (Seismic Zones 3 and 4).** In Seismic Zones 3 and 4, structural observation should be provided and noted on the plans for high rises as defined in Section 403, and for structures of Occupancy Category I, II or III as defined in Table 16-K. (Section 1702)

4. **Structural Observation Required by Building Official (Seismic Zones 3 and 4).** Structural observation should be provided (as granted authority by the building official in Section 1702, Item 5).

5. **Structural Observation (Seismic Zone 4 and N_a >1).** Structural observation shall be provided for all structures in Seismic Zone 4 that require a lateral design and have an N_a factor as set forth in Table 16-S that is greater than one except for one- and two-story buildings, housing group B, F, M, S, R-3 and U Occupancies.

II DESIGN LOADS OTHER THAN EARTHQUAKE AND WIND

II. DESIGN LOADS OTHER THAN EARTHQUAKE AND WIND

A. **Load Combinations — Strength Design and Working Stress Design.** Buildings and other structures and all portions thereof should be designed to resist the most critical effect resulting from the load combinations set forth in Sections 1612.2 or 1612.3.

B. **Reduction of Live Loads**

1. **Reduction Not Allowed.** Live loads should not be reduced for floors in places of public assembly and for live loads greater than 100 pounds per square foot (psf) (4.79 kN/m^2). (Section 1607.5)

2. **Reduction of Storage Live Loads.** For storage live loads exceeding 100 pounds per square foot (4.79 kN/m^2), no reduction should be made, except that design live loads on columns may be reduced 20 percent. (Section 1607.5)

3. **Maximum Reduction in Garages.** The live load reduction should not exceed 40 percent in garages for the storage of private pleasure cars having a capacity of not more than nine passengers per vehicle. (Section 1607.5)

C. **Floor Design**

1. **Uniform Floor Loads.** The following uniform floor loads should be addressed in the design:_____ (Table 16-A)

    a. **Concentrated Floor Loads.** The following concentrated floor loads placed upon any $2^1/_2$ feet (762 mm) square should be addressed in the design: _____ (Section 1607.3.3 and Table 16-A)

    b. **Private Garage Wheel Loads.** Garage floors supporting private or pleasure-type vehicles should be designed for a minimum concentrated 2,000 pound (8.9 kN) wheel load acting on an area of 20 square inches (12 900 mm^2) without uniform live loads. (Section 1607.3.3)

    c. **Commercial Garage Wheel Loads.** Commercial garage floors should be designed for concentrated loads of two or more loads, each 40 percent of the gross weight of the maximum-size vehicle, spaced 5 feet (1524 mm) on center. (Section 1607.3.3)

    d. **Partition Load.** The floor dead load should include a 20 psf (0.96 kN/m^2) partition load. (Section 1606.2)

    e. **Posting of Live Loads.** Plans should indicate that live loads will be conspicuously posted by the owner in accordance with Section 1607.3.5.

f.   **Openings in Floors.** Openings in floors should be accounted for in the calculations and a complete load path should be established. (Section 1605.2)

D.   Roof Design

1.   **Unbalanced Roof Loading.** Unbalanced roof loads should be used where such loading will result in larger members or connections. (Section 1607.4.3)

2.   **Minimum Snow Loads.** A minimum snow load of_____ full or unbalanced should be considered. (Section 1614)

3.   **Unbalanced Snow Loads.** Potential unbalanced accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration should be considered. (Section 1614)

The following comments apply if the alternate design procedure in Appendix Chapter 16, Division I, is used.

4.   **Single-gable Roofs.** Single-gable roofs should be designed for unbalanced snow loads in accordance with Section 1641.2.

5.   **Multiple-gable Roofs.** Multiple-gable roofs should be designed in accordance with Section 1641.3.

6.   **Roof Eave Overhang.** The roof eave overhang should be designed for a uniformly distributed load of 2.0 P_f. (Section 1643)

7.   **Exits.** Building exits under down-slope eaves should be protected from sliding snow and ice. (Section 1643)

8.   **Drifting Snow on Lower Roofs.** Drifting snow on lower roofs should be addressed in accordance with Section 1644.2. Sliding snow surcharge should be considered in accordance with Section 1644.4.

9.   **Drifting Snow on Adjacent Roofs.** Drifting snow on adjacent roofs should be addressed in accordance with Section 1644.3.

10.   **Drifting Snow at Projections.** Drifting snow at mechanical equipment, penthouses, parapets and other projections should be addressed in accordance with Section 1644.5.

11.   **Deflection.** It should be shown that the deflection of structural members supporting roofs with slopes less than $^1/_2$:12 does not exceed $^L/_{180}$ on the basis of roof snow loads plus K times dead load. (Section 1646)

DESIGN LOADS OTHER THAN EARTHQUAKE AND WIND

E. Special Loads and Other Minimum Loads

1. **Wall Design.**

   a. **Interior Walls.** Interior walls and partitions should be designed for a live load of 5 psf (0.24 kN/m^2) applied perpendicular to the wall with a deflection not exceeding $^1/_{120}$ of the span for walls with flexible finishes. (Section 1611.5)

   b. **Anchorage of Walls.** Concrete and masonry walls should be anchored to all floors, roofs and other structural elements which provide required lateral support for the wall in accordance with Sections 1605.2.3 and 1611.4. Also see Sections 1632,1633.2.8 and 1633.2.9 for earthquake design requirements.

   c. **Exterior Walls.** Exterior walls should be designed to resist wind loads as set forth in Section 1622 and seismic loads as set forth in Section 1632.

2. **Guardrails and Handrails.** Guardrails and balcony railings should be designed to withstand a 50 (20) pound per lineal foot (plf) [2.4 (1.0) kN/m^2] load applied horizontally at right angles to the top rail. Handrails should be mounted such that the support system can withstand a 200-pound (890 N) load applied in any direction at any point in the rail. Intermediate rails, panel fillers and their connections should be capable of withstanding a load of 25 psf (1.2 kN/m^2) applied horizontally. (Table 16-B and Footnotes 8, 9 and 11)

3. **Vehicle Barriers.** In parking garages where any parking area is located more than 5 feet (1524 mm) above the adjacent grade, vehicle barriers should be provided. (Section 311.2.3.5) The vehicle barrier should be able to withstand a horizontally applied load of 6,000 pounds (26.9 kN) at a height of 18 inches (457 mm). (Table 16-B and Footnote 10)

F. Soil Design Loads

1. **Equivalent Fluid Pressure.** Retaining walls should be designed for an equivalent fluid pressure of not less than 30 pounds per cubic foot (4.71 kN/m^3). (Section 1611.6)

2. **Surcharge Loads.** Surcharge loads on retaining walls should be addressed in the design. (Section 1611.6)

3. **Sliding and Overturning.** A factor of safety of 1.5 to resist sliding and overturning should be used in the design of retaining walls. (Section 1611.6)

4. **Soils Report Loads.** Recommendations for soil loads as set forth in the soils investigation report should be followed in the design. (Section 1804.3)

5. **Water Table.** The effects of the high water table should be addressed in the design. (Section 1611.8)

DESIGN LOADS OTHER THAN EARTHQUAKE AND WIND

G.   Deflection Criteria

1.   **Table 16-D Deflection Criteria.** Calculations should be provided to show compliance with the deflection criteria set forth in Table 16-D. (Section 1613)

III. WIND DESIGN

NOTE: Although the wind design forces may be greater than the earthquake design forces, detailing requirements and limitations prescribed in Division IV—Earthquake Design and the material sections should be followed. (Section 1626.3)

A. Design Formula (20-1) Coefficients

1. **Design Pressure.** The wind stagnation pressure q_s should be at least _____ as set forth in Table 16-F for a basic wind speed of _____.

2. **Exposure.** It should be verified that the exposure factor used in the calculations is appropriate for the site conditions given the definition set forth in Section 1616.

3. **Importance Factor.** An importance factor I_w = 1.15 should be used in accordance with Table 16-K.

B. Primary Frames and Systems

1. **Gabled Rigid Frames.** The normal force method (Method 1) should be used for gabled rigid frames. (Section 1621.2)

2. **Simultaneously Acting Wind Pressures.** The wind pressures determined for Method 1 using the various C_q factors in Table 16-H should be assumed to act simultaneously normal to all exterior surfaces. (Section 1621.2)

3. **C_e at Mean Roof Height.** For Method 1 pressures on roofs and leeward walls, C_e should be evaluated at the mean roof height. (Section 1621.2)

4. **Projected Area Method.** For the projected area method (Method 2), horizontal pressures should be assumed to act upon the full vertical projected area of the structure, and the vertical pressures should be assumed to act simultaneously upon the full horizontal projected area. (Section 1621.3)

C. Overturning Loads

1. **OTM $\leq$ $(^2/_3)$(DLRM).** The base overturning moment (OTM) for the entire structure and the individual primary lateral-resisting elements should be shown by calculation not to exceed two-thirds of the dead-load-resisting moment (DLRM). (Section 1621.1)

WIND DESIGN

D.   Elements and Components

1. **Table 16-H Pressure Coefficients.** Elements and components should be designed for wind loads using the pressure coefficients set forth in Table 16-H. Elements and components both "in" and "not in" areas of discontinuity should be considered. (Section 1622)

2. C_e **Value.** The value of C_e for outward acting forces should be obtained from Table 16-G based on the mean roof height and applied for the entire height of the structure. (Section 1622)

IV ⎸ EARTHQUAKE DESIGN — LOADS

IV. EARTHQUAKE DESIGN — LOADS

A.   Terms used to Calculate Earthquake Load "*E*" and Resulting Earthquake Design Forces

Z — Zone Factor (Table 16-I)

S_A, S_B, S_C, S_D, S_E or S_F — Soil Profile Types (Table 16-J)

N_a, N_v — Near-source Factors (Zone 4 only) (Tables 16-S, 16-T)

C_a, C_v — Seismic Coefficients (Soil Type and Seismic Zone) (Tables 16-S, 16-T)

R — Response Factor (Table 16-N) (*R* ≈ R_W/1.4)

V — Base Shear (Section 1630.2)

ρ — Redundancy Factor [Section 1630.1.1, Formula (30-3)]

E — Earthquake Load [Section 1630.1.1, Formula (30-1)*]

Earthquake Design Forces — LRFD or Strength Design (Section 1612.2) Allowable Stress Design (ASD) or Working Stress Design (Section 1612.3)

* For calculation of E_m, see Section 1630.1.1, Formula (30-2).

IV EARTHQUAKE DESIGN — LOADS

B. Minimum Design Lateral Forces and Related Effects

1. **LRFD Load Combinations**. Where load and resistance factor design (LRFD) (Strength Design) is used, the basic load combinations of Section 1612.2.1 are required. Factored load combinations of this section shall be multiplied by 1.1 for concrete and masonry. (Section 1612.2.1). The 1.1 factor accounts for the incompatability between the ϕ factors of American Concrete Institute (ACI) 318 and the load factors of Section 1612.2.1, Formulas (12-5) and (12-6). (Reference: S.K. Ghosh, "Design of Reinforced Concrete Buildings under the 1997 UBC," *Building Standards,*™ May-June 1998.)

2. **Allowable Stress Design Load Combinations**. Where allowable stress design (working stress design) is used, the basic load combinations of Section 1612.3.1 or 1612.3.2 shall be used. When using the load combinations of Section 1612.3.1, no increase in allowable stresses shall be permitted. (Section 1612.3)

3. **Load Combinations**. The earthquake loads as determined from Formula (30-1) shall be used in the load combinations set forth in Section 1612. (Section 1630.1.1)

4. **Determination of Reliability/Redundancy Factor ρ Seismic Zones 3 and 4**. For determination of E in Formula (30-1), the earthquake load (E_H) due to the base shear, V, as set forth in Section 1630.2 or the design lateral force, F_p, as set forth in Section 1632 shall be adjusted by a Reliability/Redundancy Factor ρ as set forth in Section 1630.1.1.

   a. The Reliability/Redundancy Factor ρ as determined from Formula (30-3) shall not be less than 1.0 and need not be greater than 1.5. (Section 1630.1.1)

   b. For special moment-resisting frames, except when used in dual systems, the layout and configuration of the frames shall be such that ρ does not exceed 1.25. (Section 1630.1.1)

   c. For determination of r_{max} (the maximum element story shear ratio), the calculation of r_i shall be based on the requirements stated in Section 1630.1.1 for (braced frames), (moment frames), (shear walls) and (dual systems).

5. **Vertical Component**. The load effect from the vertical component of the earthquake ground motion shall be considered in the design. For Strength Design, E_v= 0.5 C_a ID. (Section 1630.1.1)

C. Static Approach

1. ***Z Factor.*** The Seismic Zone factor, Z, should be at least _____. (Section 1629.4.1)

2. **Importance Factor *I* = 1.25.** An importance factor *I* of 1.25 should be used in Formulas (30-4, 30-5, 30-6 and 30-7). (Table 16-K)

3. **Twenty-five Percent of Storage Load Included in _W_.** The total seismic dead load _W_ should include 25 percent of the floor storage live load. (Section 1630.1.1, Item 1)

4. **Partition Load Included in _W_.** The total seismic dead load _W_ should include a load of not less than 10 psf (0.48 kN/m^2) for partitions. (Section 1630.1.1, Item 2)

5. **Snow Load Included in _W_.** Where design snow loads exceed 30 psf (1.44 kN/m^2), the design snow load (may be reduced) should be included in the seismic dead load _W_. (Section 1630.1.1, Item 3)

6. **Determine of C_a (Table 16-Q) and C_v (Table 16-R)**

   a. Soil Profile Type S_D shall be used when the soil properties are not known in sufficient detail to determine the soil profile type. (Section 1636.2)

   b. Soil Profile Type S_f may be present at the site as determined by (building official) (geotechnical data). (Section 1636.2)

   c. Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F. (Table 16-Q and 16-R)

   d. Determination of Near Source Factors N_A and N_V is required for a building designed for a Seismic Zone factor $Z = 0.4$. (Tables 16-Q, R, S and T)

   e. Maps of known active faults indicate that design base shear for buildings should be based on a site located (___) Km from Seismic Source Type (A) or (B). (Tables 16-S, 16-T and 16-U)

   f. In Seismic Zone 3 and 4, C_a for use in "Simplified Design Basis Shear" procedure shall be based on Soil Profile Type S_D, when the soil properties are not known in sufficient detail to determine the soil profile type. (Section 1630.2.3.2)

   g. In Seismic Zone 1, 2A and 2B, C_a for use in "Simplified Design Base Shear" procedure shall be based on Soil Profile Type S_E, when the soil properties are not known in sufficient detail to determine the soil profile type. (Section 1630.2.3.2)

7. **Minimum Design Base Shear.** The total design base shear shall not be less than "V" as determined from Formula (30-6). In addition, for Seismic Zone 4, the total base shear shall also not be less than "V" as determined from Formula (30-7). (Section 1630.2.1)

8. **Limitation on _T_ from Method B.** For strength calculations, the value of T from Method B should not be over 30 percent greater than the value of T obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zones 1, 2 and 3. (Section 1630.2.2, Item 2)

9. **Height Limit (Seismic Zones 3 and 4).** The height of the building should not exceed _____ feet (mm) given the R factor used. (Table 16-N)

10. **Combination of Structural Systems in Orthogonal Directions (Seismic Zones 3 and 4).** The value of R in the north-south (east-west) direction should not be more than the value of the bearing wall system R in the east-west (north-south) direction. (Section 1630.4.3)

11. **Combination of Systems Along the Same Axes.** For other than dual systems and shear wall-frame interactive systems in Seismic Zones 0 and 1, where a combination of systems is used in the same direction, the value of R used in that direction should not be greater than the least R of the systems. (Section 1630.4.4)

12. **_R_ = 4.5 in Wood-framed Construction.** An R factor of 4.5 should be used in the design as shear walls sheathed with materials other than wood structural panels that are being used to resist the lateral loads. (Table 16-N)

13. **Top of the Structure Force, F_t.** The concentrated force F_t at the top of the structure should be calculated in accordance with Formula (30-14). (Section 1630.5)

14. **Simplified Static Lateral Procedure Limitations.** Use of the "Simplified Static Lateral Procedure" as set forth in Section 1630.2.3 is limited to the following structures of Occupancy Category 4 or 5:

    a. Buildings of any occupancy not more than 3 stories in height excluding basements, that use light-frame construction.

    b. Other buildings not more than 2 stories in height excluding basements.

D. Dynamic Approach

1. **Dynamic Lateral-force Procedure Required**. The dynamic lateral-force procedure of Section 1631 should be used as required by Section 1629.8.4 if:

    a. The structure is 240 feet (73152 mm) or more in height (except for structures, regular or irregular, in Seismic Zone 1 or in Occupancy Category IV and V in Seismic Zone 2).

    b. The structure has a stiffness, weight or geometric vertical irregularity of Type 1, 2 or 3.

    c. The structure is in Seismic Zones 3 and 4 over five stories or 65 feet (19 812 mm) in height, not having the same structural system throughout its height except as permitted by Section 1631.2.

    d. The structure is located on Soil Profile Type S_F and has a period greater than 0.7 second.

2. **Requirements for Dynamic Analysis Procedure**. The dynamic analysis procedure should comply with the requirements of Section 1631.

E. Horizontal Distribution of Shear and Torsional Moments

1. **Justification of Center of Rigidity and Center of Mass.** The location of the assumed center of rigidity and center of mass should be identified and justified by calculation. (Section 1630.6)

2. **Horizontal Distribution of Shear.** The diaphragm should be assumed rigid unless it can be shown to be flexible in accordance with Section 1630.6. The design story shear should be distributed to the various lateral-load-resisting elements in proportion to their rigidities. (Sections 1630.6 and 1605.2.1)

3. **Five Percent Displacement of Mass.** Where diaphragms are not flexible, the assumed calculated center of mass at each level should be displaced 5 percent and increased shears resulting from the torsion should be addressed. (Sections 1630.6 and 1605.2.1)

4. **Torsional Irregularity Effects.** The accidental torsion at each level should be increased by A_x as determined from Formula (30-16) where torsional irregularity exists. (Section 1630.7)

F. Overturning

1. **Overturning Effects.** Overturning loads should be addressed in the design of lateral-load-resisting elements. (Section 1630.8) The dead load limitation in reducing overturning shall be accounted for by the use of the appropriate load combinations of Section 1612.

2. **Columns Supporting Discontinuous Elements (Seismic Zones 3 and 4).** Columns and other elements supporting discontinuous load-resisting elements should be shown to have the strength to resist the combination loads resulting from the special seismic load combinations of Section 1612.4. (Section 1630.8.2.1)

3. **Elements supporting discontinuous systems in Seismic Zones 3 and 4.** Elements shall meet the detailing and member limitation requirements of Section 1630.8.2.2.

G. $P\Delta$ Effects

1. **$P\Delta$ Effects.** $P\Delta$ effects should be considered in accordance with Section 1630.1.3.

IV EARTHQUAKE DESIGN — LOADS

H. Vertical Component

1. **Horizontal Cantilever Components (Seismic Zones 3 and 4).** Overhangs (balconies) should be designed for a net upward force of $0.7C_a Z IW_p$. (Section 1630.11)

I. Floor and Roof Diaphragms

1. **Diaphragm Design Load.** Diaphragms should be designed to resist the load set forth in Formula (33-1). (Section 1633.2.9)

2. **Minimum F_{px}.** The force F_{px} shall not be less than $0.5 C_a I w_{px}$. (Section 1633.2.9)

3. **Sub-diaphragm Maximum Length-to-Width Ratio.** The maximum length-to-width ratio of the wood structural sub-diaphragm shall be $2\frac{1}{2}$:1. (Section 1633.2.9, Item 4)

J. Elements of Structures and Nonstructural Components Supported by Structures

1. **Design Seismic Forces on Parts of Structures.** The total design lateral seismic force F_p shall be determined either from Formula (32-1) or Formula (32-2). For member and connection design, these forces shall use the load combinations or factors specified in Section 1612.2 or 1612.3. (Section 1632.2)

2. **Design Lateral Force.** Walls [equipment, ornamentation, storage racks, floor-supported cabinets and bookshelves more than 6 feet (1828 mm) in height] and their attachments should be designed to resist the loads set forth in Formula (32-1 or 32-2). (Section 1632.2, Table 16-O)

3. **Design Force for Body of Connection and Fasteners (Seismic Zones 2, 3 and 4).** The body of the connections for exterior nonbearing, nonshear wall panels should be designed for the force determined by Formula (32-2), where R_p = 3.0 and a_p = 1.0. Fasteners should be designed for the forces determined by Formula (32-2) where R_p = 1.0 and a_p = 1.0. (Section 1633.2.4.2)

4. **R_p for Anchorages.** R_p shall be taken from Table 16-O except R_p for anchorages, which shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. Anchorages constructed of nonductile material or by use of adhesive shall use an R = 1.0. (Section 1632.2)

K. Nonbuilding Structures

1. **Rigid Structures.** Rigid nonbuilding structures (those with period T less than 0.06 seconds) and their anchorages should be designed in accordance with Formula (34-1). (Section 1634.3)

2. **Other Nonbuilding Structures—Minimum _V_.** Nonbuilding structure not covered by Sections 1634.3 and 1634.4 shall have a seismic design force not less than that determined by Section 1630.2 or Formula (34-2). Additionally, for Seismic Zone 4, the base shear _V_ shall not be less than Formula (34-3). (Section 1634.5)

V | EARTHQUAKE DESIGN — SYSTEM DESIGN REQUIREMENTS

V. EARTHQUAKE DESIGN — SYSTEM DESIGN REQUIREMENTS

A. Special Requirements for Irregular Structures

1. **Vertical Irregularity.** The building has a vertical irregularity type _____ as defined in Table 16-L and compliance with the requirements listed in the reference section of the table should be demonstrated. (Section 1629.5.3)

2. **Plan Irregularity.** The building has a plan irregularity type _____ as defined in Table 16-M and compliance with the requirements listed in the reference section of the table should be demonstrated. (Section 1629.5.3)

B. Story Drift

1. **Determination of Story Drift.** Story drifts should be computed using the maximum inelastic response displacement Δ_m, where Δ_m is as determined from Formula (30-17). (Section 1630.9)

2. **Design Level Response Displacement, Δ_s.** For determination of design level response displacement, Δ_s, the following requirements of Section 1630.9.1 shall be met:

   a. Δ_s shall be calculated using the Strength Design Load combinations of Section 1612.2. (Section 1630.9.1)

   b. For reinforced concrete or masonry systems, the effects of cracked sections shall be considered. (Section 1630.1.2)

   c. For steel moment frames, panel zone deformations shall be considered. (Section 1630.1.2)

3. **Maximum Calculated Drift When T < 0.7 Second.** The calculated story drift should be shown not to exceed 0.025 times the story height. (Section 1630.10.2)

4. **Maximum Calculated Drift When $T \geq$ 0.7 Second.** The calculated story drift should be shown not to exceed 0.020 times the story height. (Section 1630.10.2)

C. Orthogonal Effects

1. **Direction of Earthquake (Seismic Zones 2, 3 and 4).** Consideration should be given to the effects of earthquake forces acting in a direction other than the principal axes. (Section 1633.1)

V EARTHQUAKE DESIGN — SYSTEM DESIGN REQUIREMENTS

D. Connection Details

1. **Design and Detailing of Connections.** Connections resisting seismic forces should be designed and detailed on the drawings. (Section 1633.2.3)

2. **Design and Detailing of Shear Transfer Connections.** Roof and floor diaphragm connections for transfer of lateral loads to vertical shear-resisting elements should be designed and detailed. (Section 1633.2.3)

3. **Design and Detailing of Drag Struts and Chord Splices.** Drag strut connections and chord splices should be designed and detailed on the drawings. (Section 1633.2.3)

E. Deformation Compatibility

1. **Framing Elements Not Part of the Lateral-load-resisting System.** Framing elements not part of the lateral-load-resisting system should be shown to be adequate for vertical-load-carrying capacity when displaced by the greater of the maximum inelastic response displacement Δ_m, considering $P\Delta$ effects determined by Section 1630.9.2, or the deformation induced by a story drift of 0.0025 times the story height. (Section 1633.2.4)

2. **Concrete and Masonry Elements that are Part of the Lateral-force-resisting System.** For concrete and masonry elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked section analysis is performed. (Section 1633.2.4)

3. **Adjoining Rigid Elements.** It should be shown that the action or failure of rigid elements enclosing (adjoining) the moment-resistant frames will not impair the vertical- and lateral-load-resisting ability of the frame. (Section 1633.2.4.1)

4. **Relative Movement between Stories.** Connection and panel joints of exterior nonbearing, nonshear wall panels or elements should be shown to allow for a relative movement between stories of not less than 2 times the story drift caused by wind, the calculated story drift based on the maximum inelastic response displacement Δ_m or $^1/_2$ inch (13 mm), whichever is greater. (Section 1633.2.4.2, Item 1)

5. **Connections Permitting Movement.** The connections for exterior nonbearing, nonshear wall panels or elements which provide for deformation compatibility should be sliding connections using slotted or oversized holes, connections which permit movement by bending of steel, or an equivalent. (Section 1633.2.4.2, Item 2)

6. **Connections Embedded in Concrete.** The connection fasteners embedded in concrete for exterior nonbearing, nonshear wall panels or elements should be attached to, or hooked around, reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. (Section 1633.2.4.2, Item 6)

V EARTHQUAKE DESIGN — SYSTEM DESIGN REQUIREMENTS

F. Ties and Continuity

1. **Tie between Portions of Building.** The smaller portion of the building should be tied to the rest of the building with elements having the strength to resist $0.5C_aI$ times the weight of the smaller portion. (Section 1633.2.5)

2. **Tie for Horizonal Force at Beams, Girders and Trusses.** Beams, girders and trusses should be positively connected to resist a horizontal force acting parallel to them not less than $0.3C_aI$ times the dead plus live load. (Section 1633.2.5)

G. Collector Elements

1. **Design and Detailing of Collector Members.** Collector members (drags) should be designed and detailed on the drawings. (Sections 1633.2.3 and 1633.2.6)

2. **Design Force For Collector Elements, Splices and Connections.** Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33-1), and, in addition, shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612.4. (Section 1633.2.6)

H. Diaphragms

1. **Maximum Diaphragm Deflection.** Diaphragm deflection in the plane of the diaphragm should not exceed the permissible deflection of the attached elements. (Section 1633.2.9, Item 1)

2. **Flexible Diaphragms Providing Lateral Support for Masonry or Concrete.** Design forces for flexible diaphragms should be calculated using Formula (33-1), based on a load determined in accordance with Section 1630.2 using an R not exceeding 4. (Section 1633.2.9, Item 3)

3. **Continuous Tie Requirement.** Diaphragms supporting concrete or masonry walls should have continuous ties between diaphragm chords to distribute the anchorage forces specified in Section 1633.2.8. (Section 1633.2.9, Item 4)

4. **Sub-diaphragm Length-to-Width Ratio.** The maximum length-to-width ratio for wood structural sub-diaphragms is $2^1/_2$:1. (Section 1633.2.9, Item 4)

5. **Anchorage and Wood.** Anchorage should not be accomplished by toenails or nails subject to withdrawal, nor should wood ledgers or framing be used in cross-grain bending or cross-grain tension. (Section 1633.2.9, Item 5)

6. **No One-third Increase.** Connections of diaphragms to the vertical elements and to collectors, and connections of collectors to the vertical elements in Seismic Zones 3 and 4 having a plan irregularity of Type 1, 2, 3 or 4 in Table 16-M, should be designed

without considering either the one-third increase or the duration of load increase considered in allowable stresses for elements resisting earthquake forces. (Section 1633.2.9, Item 6)

I. Framing Below the Base

1. **Columns Supporting Discontinuous Elements.** Columns supporting discontinuous lateral-force-resisting elements should comply with the special detailing requirements of Chapter 19 (22). (Section 1633.2.10)

2. **Columns Below the Base.** Columns below the base supporting the SMRF (STMF, MMRWF, IMRF, EBF) should comply with the special detailing requirements of Chapter 19 (22). (Section 1633.2.10)

J. Building Separations

1. **Building Separations.** The building should be shown to allow for a separation from adjacent structures of at least Δ_m as determined from formula (30-17) or, in the case of adjacent buildings on the same property, Δ_{mT} as determined from formula (33-2). When a structure adjoins a property line not common to a public way, that structure should also be set back from the property line by at least Δ_m. (Section 1633.2.11)

K. Anchorage of Concrete or Masonry Walls

1. **Anchorage to Floors and Roofs.** Concrete and masonry walls shall be anchored as required in Section 1605.2.3. Such anchorage shall be capable of resisting the wind and earthquake loading required in Chapter 16 using the load combinations of Sections 1612.2 or 1612.3, or a minimum horizontal force of 280 pounds per linear foot of wall (420 plf in Seismic Zone 4) substituted for *E*. (Sections 1605.2.3, 1611.4 and 1633.2.8)

2. **Anchorage to Flexible Diaphragms in Seismic Zones 3 and 4.** Out-of-plane wall anchorage to flexible diaphragms located in Seismic Zones 3 and 4 shall comply with the specific design requirements of Section 1633.2.8.1. (Section 1633.2.8)

3. **Embedded Straps—Seismic Zones 3 and 4.** In Seismic Zones 3 and 4, the diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. (Section 1633.2.8)

VI FOUNDATION DESIGN REQUIREMENTS

VI. FOUNDATION DESIGN REQUIREMENTS

A. Foundation Investigation

1. **Expansive Soil.** The expansive characteristics of the soil should be determined in accordance with UBC Standard 18-2. Foundations resting on soils with an expansion index greater than 20 require special design consideration. (Section 1803.2) An investigation and report should be submitted addressing provisions to be made in the foundation design and construction to safeguard against damage due to the expansiveness. (Section 1804.4)

2. **Classification of Soil.** The classification of the soil should be determined by an engineer or architect licensed by the state to practice as such. (Section 1804.1)

3. **Potential for Seismically Induced Soil Liquefaction and Soil Instability (Seismic Zones 3 and 4).** The potential for seismically induced soil liquefaction and soil instability should be evaluated as set forth in Section 1804.5. (Section 1804.2)

4. **Soil Classification and Design-bearing Capacity.** The soil classification and design-bearing capacity should be shown on the plans. (Section 1804.3)

5. **Soils Report Required.** A soils investigation report should be submitted which includes the information required by Section 1804.3.

B. Footing Requirements

1. **Limitations of Table 18-I-A.** Table 18-I-A should not be used under any one of the following conditions (Section 1805):

   a. Expansive soil.

   b. Types I, III and IV construction.

   c. Four or more stories in height.

   d. Continuous footings with loads more than 2,000 plf (29.2 kN/m).

   e. Isolated footings with loads more than 50,000 pounds (222.4 kN).

2. **Footings and Frost Line.** Footings should extend below the frost line. (Section 1806.1)

3. **Footings Supporting Wood.** Foundations supporting wood should extend at least 6 inches (152 mm) above the adjacent finish grade. (Section 1806.1)

4. **Stepped Foundations.** Where the surface of the ground slopes more than 10 percent, foundations should be level or stepped in accordance with Section 1806.4.

5. **Footings on or Adjacent to Slopes.** The placement of structures on or adjacent to slopes steeper than 1 unit vertical to 3 units horizontal (33% slope) should be in accordance with Section 1806.5.

6. **Foundation Plates.** Foundation plates in Seismic Zones 0 through 3 should be bolted to the foundation with not less than $1/2$-inch-diameter (13 mm) steel bolts embedded at least 7 inches (178 mm) into the concrete or masonry and spaced not more than 6 feet (1829 mm) apart. In Seismic Zone 4, minimum $5/8$-inch-diameter steel bolts shall be used. (Section 1806.6)

7. **Additional Requirements for Seismic Zones 3 and 4.** The following additional requirements apply in Seismic Zones 3 and 4: (Sections 1806.6 and 1806.7)

   a. Sill bolt diameter and spacing for 3-story raised wood floor buildings shall be specifically designed.

   b. Maximum 2" x 2" x $3/16$" plate washers required for each bolt.

   c. No. 4 bar, top and bottom, required in continuous footings.

C. Pile Foundations

1. **Foundation Investigation for Piles.** The foundation investigation should specifically address the following issues in accordance with Section 1807.1:

   a. Recommended pile types and installed capacities.

   b. Driving criteria.

   c. Installation procedures.

   d. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).

   e. Pile load test requirements.

2. **Interconnection of Piles.** Pile caps and caissons should be interconnected by ties. Such ties should be capable of resisting, in tension or compression, a minimum horizontal force equal to 10 percent of the larger column vertical load. (Section 1807.2)

3. **Group Action.** Consideration should be given to the reduction of allowable pile load when piles are placed in groups in accordance with Section 1807.6.

4. **Wood Piles.** The following requirements for wood piles should be met:

   a. Wood piles should be pressure treated except as noted in Section 1808.1.

b. The allowable unit stresses for round wood piles should not exceed those set forth in Chapter 23, Division III, Part I. For piles which support their own specific load and do not occur in a pile cluster, a safety factor of 1.25 should be applied to compression parallel to the grain values, and 1.30 to extreme fiber in bending values. (Section 1808.1.2)

5. **Uncased Cast-in-place Concrete Piles.** The following requirements for concrete piles cast inplace should be met:

   a. The length of the pile should be limited to not more than 30 times the average diameter except as noted in Section 1808.2.1.

   b. The concrete should have a specified compressive strength f'_c of not less than 2,500 pounds per square inch (psi) (17.24 MPa). (Section 1808.2.1)

   c. The allowable compressive stress in the concrete should not exceed $0.33f'_c$. The allowable compressive stress of reinforcement should not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.7 MPa). (Section 1808.2.2)

6. **Metal-cased Concrete Piles.** The following requirements for metal-cased concrete piles should be met:

   a. The concrete should have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa). (Section 1808.3.1)

   b. Installation notes in accordance with Section 1808.3.2 should be specified on the plans.

   c. The allowable compressive stress in the concrete should not exceed $0.33f'_c$, except as noted in Section 1808.3. The allowable compressive stress of reinforcement should not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.7 MPa).

7. **Precast Concrete Piles.** The following requirements for precast concrete piles should be met:

   a. The piles should have a specified compressive strength f'_c of not less than 3,000 psi (20.68 MPa), and should develop a compressive strength of not less than 3,000 psi (20.68 MPa) before driving. (Section 1808.4.1)

   b. Reinforcement ties for precast concrete pile longitudinal steel should be provided in accordance with Section 1808.4.2.

   c. Precast concrete piling should be designed to resist stresses induced by handling and driving as well as by loads. The allowable stresses should not exceed the values specified in Section 1808.2.2. (Section 1808.4.3)

8. **Precast Prestressed Concrete Piles.** The following requirements for precast prestressed concrete piles should be met:

   a. Precast prestressed concrete piles should have a specified compressive strength f'_c of not less than 5,000 psi (34.48 MPa) and should develop a compressive strength of not less than 4,000 psi (27.58 MPa) before driving. (Section 1808.5.1)

   b. Reinforcement for precast prestressed concrete piles should be provided in accordance with Section 1808.5.2.

   c. Allowable stresses for precast prestressed concrete piles should not exceed those set forth in Section 1808.5.3.

9. **Structural Steel Piles.** Structural steel piles should meet the requirements of Section 1808.6.

10. **Concrete-filled Steel Pipe Piles.** Concrete-filled steel pipe piles should meet the requirements of Section 1808.7.

11. **Seismic Design Requirement for Piles and Caissons (Seismic Zones 3 and 4).** The design of piles, caissons and caps should include the effects of lateral displacements. The special detailing requirements set forth in Section 1809.5.2 should be provided for a length of piles equal to 120 percent of the flexural length. (Section 1809.5.1)

VII CONCRETE

VII. CONCRETE

A.   Material Requirements

1. **Welded Reinforcement.** Reinforcement which is to be welded and the welding procedure to be used should be specified on the plans. Also, a report of material properties necessary to conform to welding procedures specified in UBC Standard 19-1 (except for A 706 steel) should be provided. (Section 1903.5.2)

2. **Freezing and Thawing Exposures.** Plans should specify that concrete exposed to freezing and thawing or deicing chemicals will be air entrained with air content in accordance with Table 19-A-1. Concrete that will be subjected to the exposures given in Table 19-A-2 should conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. (Sections 1904.2.1 and 1904.2.2)

3. **Concrete Exposed to Deicing Chemicals.** For concrete exposed to deicing chemicals, the maximum weight of the fly ash, other pozzolans, silica fume or slag that is included in the calculation of water-cementitious materials ratio should not exceed the percentages of the total weight of cementitious materials given in Table 19-A-3.

4. **Sulfate Exposure.** Concrete exposed to sulfate-containing solutions or soils should be specified on the plans to conform to the requirements of Table 19-A-4, or should be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength as set forth in Table 19-A-4. (Section 1904.3.1) Also, calcium chloride should not be specified as an admixture in concrete exposed to severe or very severe sulfate-containing solutions as defined in Table 19-A-4. (Section 1904.3.2)

5. **Specified f'_c.** Plans should show specified compressive strength of concrete f'_c for which each part of the structure is designed. (Section 1905.1.3)

6. **Cold Weather Requirements.** Plans should specify how concrete will be maintained above 50°F (10°C) and in a moist condition for at least the first seven days after placement. (Section 1905.11.1)

7. **Conduits and Pipes Embedded in Concrete.** Limitations for conduits and pipes embedded in concrete listed in Section 1906.3 should be included in the specifications.

8. **Construction Joints.** The location of construction joints should be specified on the plans in accordance with Section 1906.4.

VII CONCRETE

B.   Reinforcement Requirements

SPACING LIMITS

1. **Parallel Bars in a Layer.** The minimum clear spacing between parallel bars in a layer should be d_b, but not less than 1 inch (25 mm). (Section 1907.6.1) Where parallel reinforcement is placed in two or more layers, bars in the upper layers should be placed directly above bars in the bottom layer with clear distance between layers not less than 1 inch (25 mm). See Table VII-1. (Section 1907.6.2)

REINFORCEMENT TYPE	TYPE MEMBER	CLEAR DISTANCE
DEFORMED BARS	FLEXURAL MEMBERS	$d_b \geq 1$ IN.
	COMPRESSION MEMBERS TIED OR SPIRALLY REINFORCED	$d_b \geq 1$ IN.
PRETENSIONING TENDONS	WIRES	$4d_b$
	STRANDS	$3d_b$

For **SI:**   1 inch = 25.4 mm.

TABLE VII-1
CLEAR DISTANCE BETWEEN BARS, BUNDLES OR TENDONS

Reproduced from *Notes on ACI 318-95 Building Code Requirements*
for Reinforced Concrete with Design Applications
Courtesy of Portland Cement Association

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2. **Longitudinal Bars in Columns.** In spirally reinforced or tied reinforced compression members, the clear distance between longitudinal bars should not be less than $1.5d_b$ or 1.5 inches (38 mm). (Section 1907.6.3)

3. **Flexural Reinforcement in Walls and Slabs.** In walls and slabs other than concrete joist construction, primary flexural reinforcement should not be spaced farther apart than three times the wall or slab thickness, or 18 inches (457 mm). (Section 1907.6.5)

4. **Prestressing Tendons.** The clear distance between pretensioning tendons at each end of a member should not be less than $4d_b$ for wire or $3d_b$ for strands. (Section 1907.6.7.1)

5. **Bundled Bars.** Bundled bars should be detailed on the plans in accordance with the requirements set forth in Section 1907.6.6.

6. **Spacing Limits for Shear Reinforcement.** Spacing of shear reinforcement placed perpendicular to axis of member should not exceed $d/2$ in nonprestressed members and $(3/4)h$ in prestressed members or 24 inches (610 mm). (Section 1911.5.4.1) When V_s exceeds $4\sqrt{(f'_c)}\,b_w d$ [For **SI**: $0.33\sqrt{(f'_c)}\,b_w d$], this maximum spacing should be reduced by one-half. (Section 1911.5.4.3)

7. **Minimum Concrete Cover.** Minimum concrete cover should be specified on the plans in accordance with Section 1907.7.

> ### SPIRAL REINFORCEMENT FOR
> ### COMPRESSION MEMBERS

8. **Minimum Spiral Reinforcement.** The ratio of spiral reinforcement should not be less than $\rho_s = 0.45\left(\dfrac{A_g}{A_c} - 1\right)\dfrac{f'_c}{f_y}$. (Section 1910.9.3)

9. **Minimum Size of Spirals.** For cast-in-place construction, the size of spirals should not be less than $3/8$-inch (9.5 mm) diameter. (Section 1907.10.4.2)

10. **Clear Spacing.** Clear spacing between spirals should not exceed 3 inches (76 mm) or be less than 1 inch (25 mm). (Section 1907.10.4.3)

11. **Splices.** Splices in spiral reinforcement should be lap splices of $48d_b$, but not less than 12 inches (305 mm), or welded. (Section 1907.10.4.5)

12. **Extent of Spirals** Spirals should extend as required by Sections 1907.10.4.6, 1907.10.4.7 and 1907.10.4.8.

VII CONCRETE

13. **Minimum Size of Ties.** Ties should be at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for others. (Section 1907.10.5.1)

14. **Spacing of Ties: Seismic Zones 2, 3 and 4 for Frame Members Not Part of the Lateral-force-resisting System.** Ties should be spaced in accordance with Section 1921.7.2 or 1921.7.3 depending on the magnitude of moments induced in those members when subjected to Δ_m (defined in Section 1630.9). When induced moments under lateral displacements are not calculated, Section 1921.7.3 shall apply.

COLUMN REINFORCEMENT DETAILING

a. Members with induced moments and shears under lateral displacements of Section 1921.7.1 combined with the factored gravity moments and shear loads not exceeding the design moment and shear strength of the frame member.

   Note: (Members with factored gravity axial forces exceeding $0.3P_o$ shall have the spacing per Figure VII-1-b).

S_2

CLEARWEIGHT OF COLUMN

S_2= COLUMN TIE SPACING THROUGHOUT THE FULL HEIGHT NOT TO EXCEED 6 LONGITUDINAL BAR DIAMETERS,16 TIE DIAMETERS, ONE-HALF THE LEAST COLUMN DIMENSION OR 6 INCHES (152 mm).
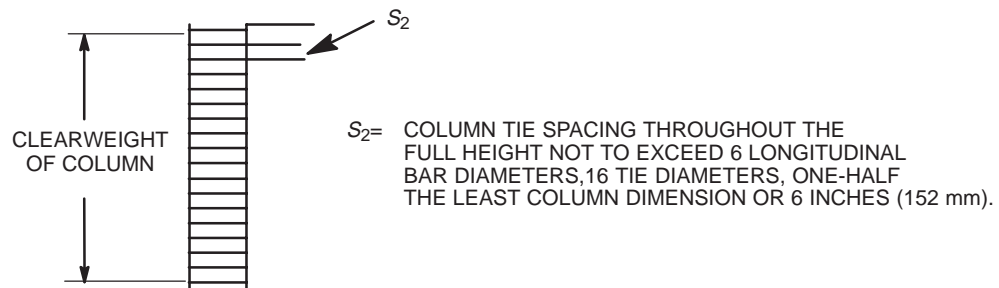
Tie Spacing, Section 1921.7.2.2
Figure VII-1-a

b. Members with induced moments and shears under lateral displacements of Section 1921.7.1 exceeding the design moment strength of the frame member or where induced moments are not calculated.

4
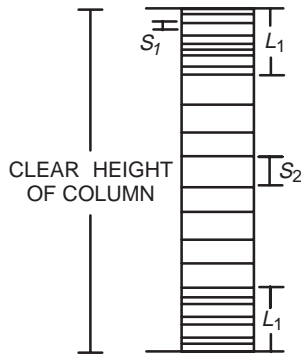
$L_1 =$ LARGEST COLUMN DIMENSION, BUT NOT LESS THAN $1/6$ THE CLEAR HEIGHT, OR 18 INCHES (457 mm).

$S_1 =$ COLUMN TIE SPACING AT TOP AND BOTTOM, NOT TO EXCEED 6 LONGITUDINAL BAR DIAMETERS, 16 TIE DIAMETERS, ONE FOURTH THE LEAST COLUMN DIMENSION, OR 4 INCHES (102 mm). THE FIRST TIE SHOULD BE WITHIN A DISTANCE EQUAL TO 0.5 S_1 FROM THE FACE OF THE JOINT AND SHOULD EXTEND THROUGH THE COLUMN-BEAM JOINT WHERE REQUIRED BY SECTION 1921.7.3.3.

$S_2 =$ COLUMN TIE SPACING NOT TO EXCEED 2 S_1 NOR 6 INCHES (152 mm).

NOTE: Ties should have hooks not less than 135 degrees with extensions not less than six tie-bar diameters or 3 inches (76 mm).

Tie Spacing, Section 1921.7.2.3 and 1921.7.3.3
Figure VII-1-b

15. **Spacing of Ties: Other.** Vertical spacing of ties should not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member. (Section 1907.10.5.2) The lateral ties should have a 135-degree-minimum turn plus an extension of at least $6d_b$ at the free end. (Sections 1907.1.3 and 1907.10.5.6)

16. **Arrangement of Ties.** Ties should be arranged such that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and a bar should be not farther than 6 inches (152 mm) clear on each side along the tie from such a laterally supported bar. (Section 1907.10.5.3)

17. **Extent of Ties.** Ties should extend as required by Sections 1907.10.5.4 and 1907.10.5.5.

18. **Shrinkage and Temperature Reinforcement in Structural Slabs.** Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement should be provided in structural slabs where the flexural reinforcement extends in one direction only in accordance with the following (Section 1907.12.1):

    a. **Deformed Reinforcement.** The area of the deformed reinforcement should not be less than that set forth in Section 1907.12.2.1. The reinforcement should not be spaced more than five times the slab thickness or 18 inches (457 mm). (Section 1907.12.2.2) Also, the reinforcement should be developed in accordance with Section 1912.1 or 1912.15.

    b. **Prestressing Tendons.** When prestressing tendons are used for shrinkage and temperature reinforcement, they should be subject to the requirements of Section 1907.12.3.

DEVELOPMENT OF REINFORCEMENT IN TENSION

19. **Development of Deformed Bars and Wire in Tension.** The development length for deformed bars and wire in tension should not be less than 12 inches (305 mm) nor the length computed by Section 1912.2.2 or 1912.2.3. The basic simplified development lengths (units = inches) set forth in Section 1912.2.2 for Grade 60 bars are as follows:

Development length l_d for Grade 60, Uncoated, Bottom Reinforcement in Normal Weight Concrete

	f'_c psi	No. 6 (#19-Metric) and smaller bars and deformed wires	No. 7 (#22-Metric) and larger bars
Clear spacing of bars being developed or spliced not less than d_b clear cover not less than d_b, and beam stirrups or column ties throughout l_d not less than the code minimum. or Clear spacing of bars being developed or spliced not less than $2d_b$ and clear cover not less than d_b.	3,000	$44d_b$	$55d_b$
	4,000	$38d_b$	$47d_b$
	5,000	$34d_b$	$42d_b$
	6,000	$31d_b$	$39d_b$
	8,000	$27d_b$	$34d_b$
	10,000	$24d_b$	$30d_b$
Other cases	3,000	$66d_b$	$82d_b$
	4,000	$57d_b$	$71d_b$
	5,000	$51d_b$	$64d_b$
	6,000	$46d_b$	$58d_b$
	8,000	$40d_b$	$50d_b$
	10,000	$36d_b$	$45d_b$

20. **Development of Bundled Bars.** The development length of individual bars within a bundle should be that for the individual bar, increased 20 percent for a 3-bar bundle and 33 percent for a 4-bar bundle. (Section 1912.4.1)

21. **Development of Standard Hooks in Tension.** The development length of standard hooks should not be less than $8d_b$ or 6 inches (152 mm), or the length computed as the product of the basic hook development length of Section 1912.5.2 and the applicable modification factors of Section 1912.5.3. The basic hook development lengths (units = inches) set forth in Section 1912.5.2 for Grade 60 bars are as follows:

Bar Size		f'_c (Normal Weight Concrete), psi		
English Unit	Metric Unit	3,000	4,000	5,000
#3	#10	8.2	7.1	6.4
#4	#13	11.0	9.5	8.5
#5	#16	13.7	11.9	10.6
#6	#19	16.4	14.2	12.7
#7	#22	19.2	16.6	14.8
#8	#25	21.9	19.0	17.0
#9	#29	24.7	21.4	19.1
#10	#32	27.8	24.1	21.6
#11	#36	30.9	26.8	23.9
#14	#43	37.1	32.1	28.7
#18	#57	49.4	42.8	38.3

Modification factors for the following conditions should be applied:

a. Bar yield strength. (Section 1912.5.3.1)
b. Concrete cover. (Section 1912.5.3.2)
c. Ties or stirrups. (Section 1912.5.3.3)
d. Excess reinforcement. (Section 1912.5.3.4)
e. Lightweight aggregate concrete. (Section 1912.5.3.5)
f. Epoxy-coated reinforcement. (Section 1912.5.3.6)

22. **Development of Welded Deformed Wire Fabric in Tension.** The development length of welded deformed wire fabric in tension should not be less than that set forth in Section 1912.7.

23. **Development of Welded Plain Wire Fabric in Tension.** The development length of welded plain wire fabric in tension should not be less than that set forth in Section 1912.8.

24. **Development of Prestressing Strand.** The development length of three- or seven-wire pretensioning strand should not be less than that set forth in Section 1912.9.

DEVELOPMENT OF REINFORCEMENT IN COMPRESSION

25. **Development of Deformed Bars in Compression.** The development length for deformed bars in compression should not be less than 8 inches (203 mm) or the length computed as the product of the basic development length and the applicable modification factors set forth in Section 1912.3. The basic development lengths (units = inches) set forth in Section 1912.3 for Grade 60 bars are as follows:

Bar Size		f'_c (Normal Weight Concrete), psi		
English Unit	Metric Unit	3,000	4,000	$\geq 4,000$
#3	#10	8.2	8.0	8.0
#4	#13	11.0	9.5	9.0
#5	#16	13.7	11.9	11.3
#6	#19	16.4	14.2	13.5
#7	#22	19.2	16.6	15.8
#8	#25	21.9	19.0	18.0
#9	#29	24.7	21.4	20.3
#10	#32	27.8	24.1	22.9
#11	#36	30.9	26.8	25.4
#14	#43	37.1	32.1	30.5
#18	#57	49.4	42.8	40.6

Modification factors for excess reinforcement and reinforcement enclosed within spirals or ties should be applied. (Section 1912.3)

26. **Development of Bundled Bars.** The development length of individual bars within a bundle should be that for the individual bar, increased 20 percent for a 3-bar bundle and 33 percent for a 4-bar bundle. (Section 1912.4.1)

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27. **Extension of Flexural Reinforcement.** Reinforcement should extend beyond the point it is no longer required to resist flexure for a distance equal to the effective depth of member or $12d_b$, whichever is greater, except at supports of simple spans and at free ends of cantilevers. (Section 1912.10.3)

28. **Additional Requirements for Positive Moment Reinforcement.** Positive moment reinforcement should be provided in accordance with the following:

    a. **Extension of Positive Reinforcement.** At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members should extend along the same face of the member into the support. In beams, this reinforcement should extend into the support at least 6 inches (152 mm). (Section 1912.11.1)

    b. **Limitations on Positive Reinforcement.** Positive moment tension reinforcement at simple supports and at points of inflection should be shown to comply with Section 1912.11.3.

29. **Additional Requirements for Negative Moment Reinforcement.** At least one-third the total tension reinforcement provided for negative moment at a support should have an embedment length beyond the point of inflection not less than effective depth of member, $12d_b$, or $^1/_{16}$ the clear span, whichever is greater. (Section 1912.12.3)

DEVELOPMENT OF WEB REINFORCEMENT

30. **Methods of Anchorage.** Ends of single leg, simple U- or multiple U-stirrups should be anchored by one of the means set forth in Section 1912.13.2.

GENERAL REQUIREMENTS FOR SPLICES OF REINFORCEMENT

31. **Maximum Bar Size.** Lap splices should not be used for bars larger than No. 11, except as provided in Sections 1912.16.2 and 1915.8.2.3. (Section 1912.14.2.1)

32. **Welded Splices.** Plans should indicate that welding of splices will conform with UBC Standard 19-1 and should specify the welding procedure to be used. (Section 1912.14.3.2)

33. **Strength of Welded Splices.** Full-welded splices should be shown to develop in tension at least 125 percent of the specified yield strength f_y of the bar. (Section 1912.14.3.3)

34. **Strength of Mechanical Connections.** Full mechanical connections should be shown to develop in tension or compression at least 125 percent of the specified yield strength of the bar. (Section 1912.14.3.4)

35. **Clearance and Coverage.** Welded splices and mechanical connections should maintain the clearance and coverage requirements of Sections 1907.6 and 1907.7. (Section 1912.14.3.6)

36. **Special Splice Requirements for Columns.** Lap splices, butt welded splices, mechanical connections or end-bearing splices should be used within the limitations of Sections 1912.17.2 through 1912.17.4. A splice should satisfy requirements for all load combinations for the column. (Section 1912.17.1)

SPLICES OF REINFORCEMENT IN TENSION

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37. **Lap Splice Length.** Plans should indicate Class B lap splices [$1.3l_d$, but not less than 12 inches (305 mm)] unless it is shown that (1) the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and (2) one-half or less of the total reinforcement is spliced within the required lap length, in which case a Class A lap splice (l_d) may be specified. (Sections 1912.15.1 and 1912.15.2)

38. **Welded Splices and Mechanical Connections.** Welded splices and mechanical connections used where area of reinforcement provided is at least twice that required by analysis should comply with Section 1912.15.4.

39. **Welded Wire Fabric.** Splices of welded deformed wire fabric and plain wire fabric in tension should comply with the requirements of Sections 1912.18 and 1912.19.

SPLICES OF REINFORCEMENT IN COMPRESSION

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40. **Lap Splice Length.** Compression lap splice lengths for Grade 60 bars should be not less than 12 inches (305 mm) or the following, in accordance with Section 1912.16.1:

Lap bar splice enclosed within ties having an effective area not
less than 0.0015_{hs} (Section 1912.17.2.4) . . . . $25d_b$
Lap bar splice enclosed within spirals (Section 1912.17.2.5) . . . . $22.5d_b$
Other conditions . . . . $30d_b$

41. **Bars of Different Sizes.** When bars of different sizes are lap spliced in compression, the splice length should be the larger of the development length of the larger bar or splice length of the smaller bar. Bar sizes No.14 and 18 may be lap spliced to No. 11 and smaller bars. (Section 1912.16.2)

VII CONCRETE

C. Design Requirements

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GENERAL

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1. **Load Considerations.** Consideration should be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete and unequal settlement of supports. (Section 1908.2.4)

2. **Modulus of Elasticity.** The Modulus of elasticity of concrete should be determined in accordance with Section 1908.5.

3. **Load Combinations.** The following load combinations should be investigated. (Section 1909.2):

$U = 1.4D + 1.7L$	Formula (9-1)
$U = 0.75 (1.4D + 1.7L + 1.7W)$	Formula (9-2)
$U = 0.9D + 1.3W$	Formula (9-3)
$U = 1.4D + 1.7L + 1.7H$	Formula (9-4)
$U = 0.75 (1.4D + 1.4T + 1.7L)$ where	
$U \geq 1.4 (D + T)$	Formulas (9-5) and (9-6)

   For earthquake loading, See Section 1909.2.3 and 1612.2.1, Exception 2

$U = 1.1[1.2D + 1.0 E + (f_1 L + f_2 S)]$	(12-5)
$U = 1.1[0.9D \pm (1.0 E)]$	(12-6)

4. **Strength-Reduction Factors.** Strength-reduction factors should be taken as set forth in Section 1909.3. Note that the shear strength-reduction factor in Seismic Zones 3 and 4 should be 0.6 for the design of walls, topping slabs used as diaphragms over precast concrete members and structural framing members, with the exception of joints, if their nominal shear strength is less than the shear corresponding to development of their nominal flexural strength. (Section 1909.3.4.1)

5. **Bearing Strength.** The design bearing strength on concrete should not exceed (ϕ) ($0.85f'_c A_1$) except when the supporting surface is wider on all sides than the loaded area, in which case design bearing strength on the loaded area shall be multiplied by $\sqrt{(A_2/A_1)}$ but not more than 2. (Section 1910.17.1)

6. **Reduction in Shear and Torsional Moment Strength for Lightweight Concrete.** Shear strength and torsional moment strength for lightweight concrete should be reduced in accordance with Section 1911.2.

7. **Shear Strength for Nonprestressed Members.** The shear strength for nonprestressed members should be calculated in accordance with Section 1911.3.

8. **Design of Shear Reinforcement.** Where factored shear force V_u exceeds shear strength $(\phi) V_c$, shear reinforcement should be designed and provided in accordance with Section 1911.5.

9. **Combined Shear and Torsion Strength for Nonprestressed Members.** Reinforcement required for torsion shall be added to that required for the shear, moment and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement must be met. (Section 1911.6.3.8)

10. **Shear-friction.** Where direct shear is being transferred across a given plane, a shear-friction analysis in accordance with Section 1911.7 should be provided.

11. **Special Shear Design Provisions for Brackets and Corbels.** Brackets and corbels with a shear span-to-depth ratio *a/d* not greater than unity and subject to a horizontal tensile force N_{uc} not larger than V_u shall comply with the design requirements of Section 1911.9.

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FLEXURAL MEMBER DESIGN

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12. **Span Lengths.** Span lengths of members not built integrally with supports should be considered the clear span plus depth of member, but need not exceed the distance between centers of supports. In analysis of frames or continuous construction for determination of moments, span length should be taken as the distance center to center of supports. (Section 1908.7)

13. **Limitations on Methods of Analysis.** The approximate moments and shears set forth in Section 1908.3.3 may be used in the design of continuous beams provided the following conditions exist:

    a. There are two or more spans;
    b. Spans are approximately equal, with the longer of two adjacent spans not greater than the shorter by more than 20 percent;
    c. Loads are uniformly distributed;
    d. Unit live load does not exceed three times unit dead load; and
    e. Members are prismatic.

14. **Minimum Reinforcement.** The minimum reinforcement ratio should not be less than

    $3\dfrac{\sqrt{f'_c}}{f_y}\ b_w d$ Alternatively, area of reinforcement provided at every section should be

    at least one-third greater than that required by analysis. (Section 1910.5.3)

15. **Maximum Reinforcement.** When the design axial load strength is less than the smaller of $0.10f'_c A_g$ or $(\phi) P_b$, the ratio of reinforcement provided should not exceed

0.75 of the reinforcement ratio producing balanced strain conditions for the section under flexure without axial load. (Section 1910.3.3)

16. **Distribution of Flexural Reinforcement.** When the design yield strength f_y for tension reinforcement exceeds 40,000 psi (275.8 MPa), the reinforcement should comply with the distribution requirements of Section 1910.6.4.

17. **Deep Flexural Members.** The following special provisions should be complied with for deep flexural members:

    a. When the depth of the web exceeds 3 feet (914 mm), longitudinal skin reinforcement should be provided in accordance with Section 1910.6.7. Also, when the overall depth-to-clear-span ratio is greater than two-fifths for continuous spans or four-fifths for simple spans, the requirements of Section 1910.7 should be met. (Section 1910.7)

    b. For members with l_n/d less than 5 that are loaded on one face and supported on the opposite face so that the compression struts can develop between the loads and the supports, the shear requirements of Section 1911.8 should be met.

18. **Maximum Distance Between Lateral Supports.** Spacing of lateral supports for a beam should not exceed 50 times the least width b of compression flange or face. (Section 1910.4.1)

19. **Minimum Shear Reinforcement.** Minimum shear reinforcement should be provided in reinforced flexural members where the factored shear force V_u exceeds one-half the shear strength provided by concrete, except as noted in Section 1911.5.5.1. Minimum area of shear reinforcement should be calculated in accordance with Section 1911.5.5.

20. **Control of Deflections.** Reinforced concrete members subject to flexure should be designed to have adequate stiffness to limit deflections or any deformations that affect strength or serviceability of a structure adversely. (Section 1909.5.1)

COMPRESSION MEMBER DESIGN

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21. **Unbalanced Loads.** In frames or continuous construction, consideration should be given to the effect of unbalanced floor or roof loads on both exterior and interior columns, and of eccentric loading due to other causes. (Section 1908.8.2)

22. **Maximum Reinforcement.** When the design axial load strength is less than the smaller of $0.10f'_c A_g$ or $(\phi) P_b$, the ratio of reinforcement provided should not exceed 0.75 of the reinforcement ratio producing balanced strain conditions for the section under flexure without axial load. (Section 1910.3.3)

23. **Maximum Design Axial Load Strength.** The design axial load strength should not be taken greater than the following (Section 1910.3.5):

    a. For nonprestressed members with spiral reinforcement:

$$\phi P_{n(max.)} = 0.85\phi[0.85f'_c (A_g - A_{st}) + f_y A_{st}]$$ Formula (10-1)

    b. For nonprestressed members with tie reinforcement:

$$\phi P_{n(max.)} = 0.80\phi[0.85f'_c (A_g - A_{st}) + f_y A_{st}]$$ Formula (10-2)

24. **Reinforcement Requirements.** The area of longitudinal reinforcement for noncomposite compression members should not be less than 0.01 or more than 0.08 times gross area A_g of the section. Also, the minimum number of longitudinal bars in compression members should be four for bars within rectangular or circular ties, three for bars within triangular ties, and six for bars enclosed by spirals. (Section 1910.9)

25. **Consideration of Slenderness Effects.** The design of compression members shall be based on the factored forces and moments from a second-order analysis considering materials nonlinearity and cracking as well as the effects of member curvature and drift, shrinkage and creep, interaction with supporting foundation and the effects of duration of loads. In lieu of such an analysis, slenderness effects in compression members may be evaluated in accordance with the procedure presented in Section 1910.11. (Section 1910.10)

26. **Column $f'_c > (1.4) (Floor f'_c)$.** When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, the requirements of Section 1910.15 should be met.

27. **Transfer of Moments to Columns.** Transfer of moment at connections of framing elements to columns should be considered in accordance with Section 1911.11.

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WALL DESIGN

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28. **Design for Shear.** Design for shear forces perpendicular to face of the wall should be in accordance with provisions for slabs in Section 1911.12. Design for horizontal shear forces in plane of wall should be in accordance with Sections 1911.10.2 through 1911.10.8.

29. **Effective Area for Concentrated Loads.** The horizontal length of wall to be considered as effective for each concentrated load should not exceed center-to-center distance between loads, or width of bearing plus four times the wall thickness. (Section 1914.2.4)

30. **Minimum Reinforcement.** Minimum vertical and horizontal reinforcement should be in accordance with Sections 1914.3.2 and 1914.3.3 unless a greater amount is required for shear by Sections 1911.10.8 and 1911.10.9.

31. **Minimum Thickness of Nonbearing Walls.** Nonbearing walls should be of a thickness not less than 4 inches (102 mm) or $^1/_{30}$ the least distance between members that provide lateral support. (Section 1914.6.1)

SLAB SYSTEMS DESIGN

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32. **Limitations on Methods of Analysis.** The approximate moments and shears set forth in Section 1908.3.3 may be used in the design of continuous beams provided the following conditions exist:

    a. There are two or more spans.

    b. Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent.

    c. Loads are uniformly distributed.

    d. Unit live load does not exceed three times unit dead load.

    e. Members are prismatic.

33. **T-beam Effective Slab Width.** The effective slab width for the assumed T-beam flange should not exceed the limitations set forth in Section 1908.10.

34. **Required Reinforcement in T-beam Flange.** Where primary flexural reinforcement in a slab in a T-beam flange is parallel to the beam, reinforcement perpendicular to the beam should be provided in the top of the slab in accordance with Section 1908.10.5.

35. **Minimum Thickness for One-way Slabs.** One-way slabs should be at least the minimum thickness set forth in Table 19-C-1 unless it can be shown by calculation that a lesser thickness may be used without adverse effects. (Section 1909.5.2.1) Calculated deflections should not exceed those set forth in Table 19-C-2.

36. **Minimum Thickness for Two-way Slabs.** Two-way slabs should be at least the minimum thickness determined in accordance with Section 1909.5.3.

37. **Minimum Slab-on-grade Thickness.** The minimum thickness of concrete floor slabs supported directly on the ground should not be less than $3^1/_2$ inches (89 mm). (Section 1900.4.4)

38. **Design for Shear.** Shear in slabs should be designed in accordance with Section 1911.12.

39. **Design of Two-way Slab Systems.** Two-way slab systems should be designed in accordance with Section 1913.

VII CONCRETE

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FOOTING DESIGN

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40. **Design for Shear.** Shear in footings should be designed in accordance with Sections 1911.12 and 1915.5.

41. **Moment in Footing.** The maximum factored moment for an isolated footing should be computed in accordance with Sections 1915.4.1 and 1915.4.2.

42. **Distribution of Reinforcement.** In one-way footings, two-way square footings and the long direction of two-way rectangular footings, reinforcement should be distributed uniformly across the entire width of the footing. Reinforcement in the short direction of the two-way rectangular footing should be distributed in accordance with the requirements of Section 1915.4.4.2. (Sections 1915.4.3 and 1915.4.4)

43. **Minimum Footing Depth.** The depth of footing above the bottom reinforcement should not be less than 6 inches (152 mm) for footings on soil and not less than 12 inches (305 mm) for footings on piles. (Section 1915.7)

44. **Bearing on Concrete.** Bearing on concrete at the contact surface between supported and supporting member should not exceed concrete bearing strength for either surface as given by Section 1910.15. (Section 1910.17)

45. **Transfer of Force at Base of Column, Wall or Reinforced Pedestal.** Forces and moments at the base of column, wall, or pedestal should be transferred to the supporting pedestal or footing by bearing on concrete and by reinforcement, dowels and mechanical connectors in accordance with Section 1915.8.1. In cast-in-place construction, the reinforcement required to satisfy this provision should be provided either by extending the longitudinal bars into supporting pedestal or footing, or by dowels as required by Section 1915.8.2. In precast construction, reinforcement required to satisfy this requirement may be provided by anchor bolts or suitable mechanical connectors in accordance with Section 1915.8.3.

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PRECAST CONCRETE DESIGN

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46. **Applicable Provisions.** The provisions of the code should apply to structures incorporating precast concrete members except as specifically modified in Section 1916.

47. **Details.** Details of reinforcement, connections, bearing seats, inserts, anchors, concrete cover, openings, lifting devices, fabrication and erection tolerances should be shown on the shop drawings. (Section 1916.2.4)

VII CONCRETE

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COMPOSITE CONCRETE DESIGN

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48. **Deflection Criteria.** The deflection criteria set forth in Section 1909.5.5 should be considered for shored and unshored composite construction.

49. **Composite Compression Members.** Composite compression members should comply with the requirements of Section 1910.16.

50. **Applicable Provisions.** The provisions of the code should apply to composite concrete flexural members, except as specifically modified in Section 1917. (Section 1917.1.2)

51. **Transfer of Horizontal Shear Forces.** Full transfer of horizontal shear forces should be assured at contact surfaces of interconnected elements in accordance with Section 1917.5.1.

52. **Ties for Horizontal Shear.** When ties are provided to transfer horizontal shear, the tie spacing should not exceed four times the least dimension of supported element or 24 inches (610 mm). (Section 1917.6.1)

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PRESTRESSED CONCRETE DESIGN

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53. **Deflection Criteria.** The deflection of prestressed concrete should be shown not to exceed the limits stipulated in Table 19-C-2, with immediate and long-time deflection considered as set forth in Section 1909.5.4.

54. **Maximum Design Axial Load Strength.** The design axial load strength should not be taken greater than 0.85 (for members with spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial load strength at zero eccentricity. (Section 1910.3.5.3)

55. **Shear Strength.** Shear strength provided by concrete for prestressed members should be calculated in accordance with Section 1911.4.

56. **Permissible Stresses in Concrete—Flexural Members.** Stresses in concrete immediately after prestress transfer should not exceed those set forth in Section 1918.4.1. Stresses in concrete at service loads (after allowance for all prestress losses) should not exceed those set forth in Section 1918.4.2.

57. **Permissible Stress in Prestressing Tendons.** Tensile stresses in prestressing tendons should not exceed those set forth in Section 1918.5.

58. **Loss of Prestress.** It should be shown that the sources of prestress loss listed in Section 1918.6 have been considered in the determination of the effective prestress f_{se}.

59. **Minimum Bonded Reinforcement**. A minimum area of bonded reinforcement should be provided in flexural members with unbounded prestressing tendons in accordance with Section 1918.9.

60. **Reinforcement Requirements for Prestressed Compression Members.** Members with average prestress f_{pc} less than 225 psi (1.55 MPa) should have minimum reinforcement in accordance with Sections 1907.10, 1910.9.1 and 1910.9.2 for columns, or Section 1914.3 for walls. Also, except for walls, members with average prestress f_{pc} equal to or greater than 225 psi (1.55 MPa) should have prestressing tendons enclosed by spirals or lateral ties in accordance with Section 1918.11.2.2. (Section 1918.11.2)

61. **Protection of Tendon Anchorage Zones.** Protection for tendon anchorage zones should be provided in accordance with Section 1918.13.

SHELLS AND FOLDED PLATES DESIGN

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62. **General Comment.** The design and construction of shells and folded plates should be in accordance with Section 1919.

SHOTCRETE

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63. **Applicable Provisions.** The provisions of the code should apply to shotcrete, except as specifically modified in Section 1924. (Section 1924.1)

64. **Maximum Aggregate Size.** Plans should indicate that the maximum size of coarse aggregate shall not exceed $^3/_4$ inch (19 mm). (Section 1924.3)

65. **Reinforcement Requirements.** The following reinforcement requirements of Section 1924.4 should be met:

    a. **Maximum Size.** The maximum size of reinforcement should be No. 5 bars unless it can be demonstrated by preconstruction tests that adequate encasement of larger bars can be achieved.

    b. **Minimum Clearance between Parallel Bars.** When No. 5 or smaller bars are used, there should be a minimum clearance between parallel bars of $2^1/_2$ inches

(64 mm). When bars larger than No. 5 are permitted, there should be a minimum clearance of $6d_b$.

c. **Two Curtains of Steel.** When two curtains of steel are provided, the curtain nearest the nozzle should have a minimum spacing equal to $12d_b$ and the remaining curtain should have a minimum spacing $6d_b$.

d. **Lap Splices** Lap splices should be performed by the noncontact lap splice method with at least 2 inches (51 mm) clearance between bars except as noted in this section.

e. **No Shotcrete with Spirally Tied Columns** Shotcrete should not be applied to spirally tied columns.

66. **Curing.** Plans should indicate that during the curing periods, shotcrete will be maintained above 40°F (4.4°C) and in moist condition. Also, initial and final curing conditions should be in accordance with Section 1924.9.

67. **Strength Tests.** Plans should indicate that strength tests will be administered in accordance with Section 1924.10.

68. **Visual Examination.** Plans should indicate that three 3-inch (76 mm) cores will be taken for visual examination from three areas, selected by the design engineer, which represent the worst congestion of reinforcing bars. (Section 1924.11.2)

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PLAIN CONCRETE DESIGN

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69. **Not Permitted in Seismic Zones 2, 3 and 4.** Structural plain concrete should not be used in Seismic Zones 2, 3 and 4 except as noted in Section 1922.10.3.

70. **Control Joints.** Control joints should be shown on the plans in accordance with Section 1922.3.

71. **Design Method.** Structural plain concrete members shall be designed for adequate strength in accordance with provisions of Chapter 19, using load factors as specified in Section 1909.2 and the requirements of Section 1922.5 for strength design. (Section 1922.4)

72. **Cross Section Height.** When computing strength of concrete cast against soil, the overall thickness h should be taken as 2 inches (51 mm) less than actual thickness. (Section 1922.4.8)

73. **Plain Concrete Walls.** The minimum thickness of bearing walls should not be less than $1/24$ the unsupported height or length, whichever is less, and not less than $5^1/_2$ inches (140 mm). The thickness of exterior basement walls and foundation walls

should not be less than $7^1/_2$ inches (191 mm). Not less than two No. 5 bars should be provided around window and door openings and extend at least 24 inches (610 mm) beyond the corners of openings. (Section 1922.6.6)

74. **Plain Concrete Footings.** The overall thickness of plain concrete footings should not be less than 8 inches (203 mm). Plain concrete should not be used for footings on piles. (Section 1922.7)

75. **Plain Concrete Pedestals.** The ratio of unsupported height to average least lateral dimension of plain concrete pedestals should not exceed 3. (Section 1922.8.2)

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ANCHOR DESIGN

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76. **Design Method.** Bolts and headed stud anchors cast in concrete should be shown to comply with Section 1923.1, Service Load Design, or Section 1923.2, Strength Design.

77. **Strength Design Requirements.** The following strength design requirements of Sections 1923.2 and 1923.3 should be met:

a. **Multiplication Factors.** In addition to the load factors in Section 1909.2, the following multiplication factors should be used:

   Anchor NOT embedded in tension zone
   No special inspection 2.0
   Special inspection provided 1.3

   Anchor embedded in tension zone
   No special inspection 3.0
   Special inspection provided 2.0

b. **Groups of Anchors.** For groups of anchors, the concrete design shear strength should be taken as the smallest of:

   (1) The design strength of the weakest anchor times the number of anchors,

   (2) The design strength of the row of anchors nearest the free edge in the direction of shear times the number of rows, or

   (3) The design strength of the row farthest from the free edge in the direction of shear.

c. **Reinforcement for Concrete in Tension.** Reinforcement sufficient to carry the load should be provided to prevent failure of the concrete in tension for the following cases:

    (1) For shear loading toward an edge less than 10 diameters away.

    (2) For tension or shear not toward an edge less than 4 diameters away.

d. **Minimum Edge Distance.** In no case should the minimum edge distance be less than four diameters.

e. **Interaction Formula.** When tension and shear act simultaneously, the equations of Section 1923.3.4 should be met.

D. Seismic Design Requirements

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GENERAL

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1. **Stiffness of Concrete Elements that are Part of the Lateral-force-resisting System.** For concrete elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. (Section 1633.2.4)

2. **Requirement for SMRF's in Seismic Zones 3 and 4.** Concrete frames required by design to be part of the lateral-force-resisting system in Seismic Zones 3 and 4 should be special moment-resisting frames. (Section 1633.2.7, Item 1)

3. **Requirement for IMRF's in Seismic Zone 2.** Concrete frames required by design to be part of the lateral-force-resisting system in Seismic Zone 2 should be, as a minimum, intermediate moment-resisting frames (IMRF). (Section 1633.2.7, Item 2)

4. **Anchorage of Concrete Walls.** Concrete walls should be anchored to all floors and roofs which provide out-of-plane lateral support for the wall. The anchorage should provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in Section 1611.4, 1632 or 1633.2.8. (Section 1633.2.8)

5. **Diaphragm Ties.** Diaphragms supporting concrete walls should have continuous ties or struts between diaphragm chords to distribute the anchorage forces. (Section 1633.2.9, Item 4)

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GENERAL REQUIREMENTS (SEISMIC ZONES 3 AND 4)

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6. **Diaphragm to Wall Anchorage.** In Seismic Zones 3 and 4, the diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. (Section 1633.2.8)

7. **Rigid Members Not a Part of the Lateral-force-resisting System (Seismic Zones 3 and 4).** Rigid members assumed not to be a part of the lateral-force-resisting system may be used, provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members which are not a part of the lateral-force-resisting system should also be considered. (Section 1921.2.2.2)

8. **Concrete Members Below Base of Structure (Seismic Zones 3 and 4).** Structural members below the base of the structure required to transmit to the foundation forces resulting from earthquake effects should also comply with the requirements of Section 1921. (Section 1921.2.2.3)

9. **Structural Members Assumed Not a Part of the Lateral-force-resisting System (Seismic Zones 3 and 4).** Structural members assumed not to be part of the lateral force-resisting system should comply with the requirements of Section 1921.7. (Section 1921.2.2.4)

10. **Concrete in Members Resisting Earthquake-induced Forces (Seismic Zones 3 and 4).** Compressive strength f'_c should not be less than 3,000 psi (20.69 MPa) except that footings of buildings three stories or less may have concrete with f'_c of not less than 2,500 psi (17.24 MPa). Compressive strength f'_c of lightweight-aggregate concrete used in design should not exceed 4,000 psi (27.58 MPa) except as noted in Section 1921.2.4.2.

11. **Reinforcement in Members Resisting Earthquake-induced Forces (Seismic Zones 3 and 4).** Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary elements should comply with low alloy A 706 except as noted in Sections 1921.2.5.2 through 1921.2.5.5.

12. **Welded Splices and Mechanically Connected Reinforcement (Seismic Zones 3 and 4).** Reinforcement resisting earthquake-induced flexural or axial forces in frame members or in wall boundary members may be spliced using welded splices or mechanical connectors conforming to Section 1912.14.3.3 or 1912.14.3.4 in accordance with Section 1921.

SMRF REQUIREMENTS — FLEXURAL MEMBERS (SEISMIC ZONES 3 AND 4)

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13. **Restrictions on the Design of Flexural Members (Seismic Zones 3 and 4).** Flexural members should comply with the following limitations set forth in Section 1921.3.1:

    a. **Maximum Axial Load.** The factored axial compressive force on the member should not exceed ($A_g f'_c / 10$).

    b. **Minimum Clear Span.** The clear span should not be less than four times its effective depth.

c. **Minimum Width-to-depth Ratio.** The width-to-depth ratio should not be less than 0.3.

d. **Width Restrictions.** The width should not be less than 10 inches (254 mm) or more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.

14. **Longitudinal and Transverse Reinforcement (Seismic Zones 3 and 4).** The longitudinal and transverse reinforcement requirements of Section 1921.3.2 and 1921.3.3, as illustrated in Figure VII-2, should be clearly detailed on the plans.

> **SMRF REQUIREMENTS — MEMBERS RESISTING BEND-
> ING AND AXIAL LOADS (SEISMIC ZONES 3 AND 4)**

15. **Restrictions on the Design of Members Resisting Bending and Axial Loads (Seismic Zones 3 and 4).** Members resisting bending and axial loads should comply with the following limitations set forth in Section 1921.4.1.

a. **Minimum Cross-sectional Dimension.** The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, should not be less than 12 inches (305 mm).

b. ***b/h* ratio.** The ratio of the shortest cross-sectional dimension to the perpendicular dimension should not be less than 0.4.

16. **Strong Column-Weak Beam Relationship (Seismic Zones 3 and 4).** The flexural strengths of the columns should be shown to satisfy Formula (21-1):

$$\sum M_e \ \geq \ \left(^6/_5\right) \ \sum M_g$$

If Section 1921.4.2.2 is not satisfied at a joint, columns supporting reactions from that joint should be provided with transverse reinforcement as specified in Section 1921.4.4 over their full height. (Section 1921.4.2.3)

17. **Longitudinal and Transverse Reinforcement (Seismic Zones 3 and 4).** The longitudinal and transverse reinforcement requirements of Sections 1921.4.3 and 1921.4.4, as illustrated in Figures VII-3 and VII-4, should be clearly detailed on the plans.

18. **Additional Ties around Anchor Bolts (Seismic Zones 3 and 4).** Additional ties which engage at least four vertical column bars should be provided around anchor bolts which are set in the top of a column. Such ties shall be in accordance with Sections 1921.4.4.8 and 1907.1.3, Item 3, and should be within 5 inches (127 mm) of the top of the column and consist of two No. 4 or three No. 3 bars. (Section 1921.4.4.2)

19. **Restrictions on Use of Precast Lateral-force-resisting Systems.** Precast lateral-force-resisting systems shall emulate the behavior of monolithic reinforced concrete construction and satisfy Section 1921.2.2.5. (Section 1921.2.1.6)

   a. **Frames.** Precast structural systems using frames and emulating the behavior of monolithic reinforced concrete construction shall satisfy either one of the following in accordance with Section 1921.2.2.5:

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

      2. Precast structural systems not meeting Section 1921.2.2.6 shall utilize strong connections resulting in nonlinear response away from connections. Design shall satisfy the requirements of Section 1921.2.7 in addition to all the applicable requirements of monolithic concrete construction for resisting seismic forces, except that provisions of Section 1921.3.1.2 shall apply to the segments between nonlinear action locations.

   b. **Emulation of Monolithic Construction using Strong Connections.** Members resisting earthquake-induced forces in precast frames using strong connections shall satisfy the following in accordance with Section 1921.2.7: (See Detail 2.7.1)

      1. Location of nonlinear action region shall be in accordance with Section 1921.2.7.1.

      2. Anchorage and splicing of reinforcement in the nonlinear action region shall meet the requirements of Section 1921.2.7.2.

      3. Design strength of strong connections shall satisfy Formula (21-1).

      4. Column-to-column connections shall meet the requirements of Section 1921.2.7.4.

      5. Column-face connections shall comply with the requirements of Section 1921.2.7.5.

Terminate all required top and bottom bars at the far face of the column core, providing min. distances, l_{dh} or l_d for tension per Section 21.6.4 or compression per Section 12.3, ACI 318.

Longitudinal Reinforcement, Top and Bottom
Minimum $A_s \geq 200\ b_w\ d/f_y$
Maximum $p \leq 0.025$
Maximum moment strength $\geq 25\%$ Maximum moment strength of face of either joint
Minimum of 2 bars, continuous T & B

COL. CORE

l_2

SPLICE*

l_2 l_2

EXTEND l_d STRAIGHT OR l_{dh} WHERE HOOKED INTO CORE

d

h

l_1 l_1 2 IN. MAX (51mm) l_1

SPLICE*

2 IN. MAX. (51 mm)

HOOPS S_1 STIRRUPS S_2 HOOPS S_1 MIN. 2h FROM FACE OF JOINT

l_n l_n

Engineer must provide dimensions l_1, l_2, S_1, S_2, hoop and stirrup spacing, anchorage length, cut-off points of discontinuous bars, l_d, or l_{dh} if less than across column core.

$l_n \geq$ $4d$
$l_1 =$ distance required by design for moment plus anchorage length
$l_2 =$ distance to point of inflection plus anchorage length
$d =$ design depth for -M and +M

Maximum Hoop/Tie Spacings
In length S_1, spacing for hoops $\leq d/4$; 8d_b of smallest bar; 24d_b of hoop; or 12 in. (305 mm)
*At lap splices, spacing of hoops $\leq d/4$ but not greater than 4 in. (102 mm)

In length S_2, spacing stirrups $\leq d/2$

ALTERNATE SIDE OF CROSSTIE 90° HOOKS, EXCEPT AT SPANDRELS

PLACE CROSSTIE 90° HOOKS ON SLAB-CONFINED SIDE (ANCHORED SLAB BARS NOT SHOWN)

135°
6d_b, 3 IN. MIN.
I.D. = 4 d_b
d_b

135°
6d_b
3 IN. MIN.

CROSSTIE
135°
6d_b
3 IN. MIN.

135°
6d_b
3 IN. MIN.

A →   ← A
C →   ← C
B

EXAMPLES OF OVERLAPPING HOOPS   **SPANDREL BEAM**   **DETAIL-A**   **DETAIL-B**   **DETAIL-C**
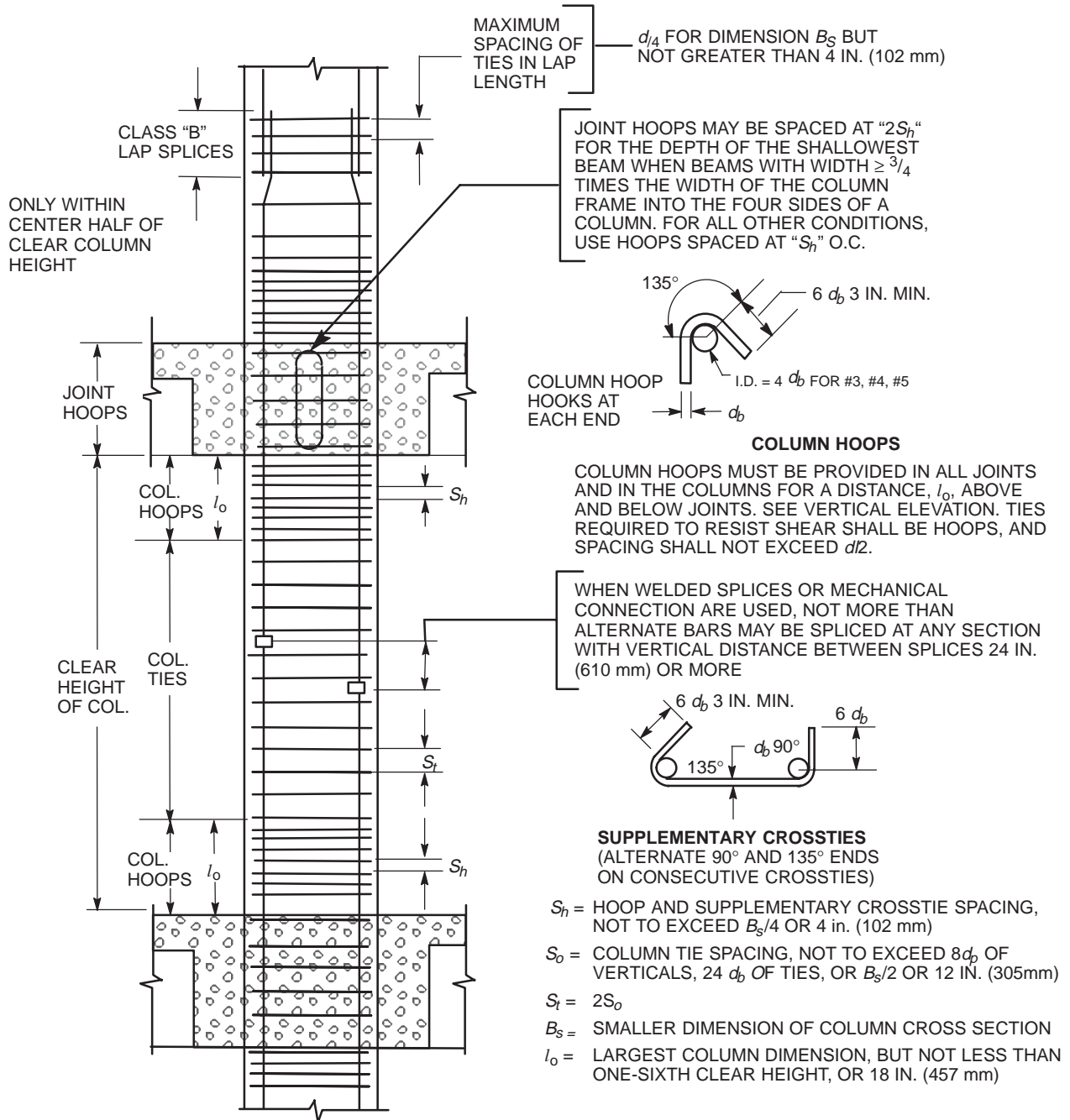
Stirrups required to resist shear shall be hoops. Throughout the length of flexural members where hoops are not required, stirrups shall be spaced at no more than $d/2$.

**FIGURE VII-2
LONGITUDINAL AND TRANSVERSE REINFORCEMENT
FOR FLEXURAL MEMBERS OF SMRFs**

Reproduced from *ACI Detailing Manual (1994)*
Courtesy of American Concrete Institu*te*

MAXIMUM SPACING OF TIES IN LAP LENGTH

$d_l/4$ FOR DIMENSION B_S BUT NOT GREATER THAN 4 IN. (102 mm)

CLASS "B" LAP SPLICES

ONLY WITHIN CENTER HALF OF CLEAR COLUMN HEIGHT

JOINT HOOPS MAY BE SPACED AT "2S_h" FOR THE DEPTH OF THE SHALLOWEST BEAM WHEN BEAMS WITH WIDTH ≥ $^3/_4$ TIMES THE WIDTH OF THE COLUMN FRAME INTO THE FOUR SIDES OF A COLUMN. FOR ALL OTHER CONDITIONS, USE HOOPS SPACED AT "S_h" O.C.

135°

6 d_b 3 IN. MIN.

JOINT HOOPS

COLUMN HOOP HOOKS AT EACH END

I.D. = 4 d_b FOR #3, #4, #5

d_b

COLUMN HOOPS

COL. HOOPS l_o

S_h

COLUMN HOOPS MUST BE PROVIDED IN ALL JOINTS AND IN THE COLUMNS FOR A DISTANCE, l_o, ABOVE AND BELOW JOINTS. SEE VERTICAL ELEVATION. TIES REQUIRED TO RESIST SHEAR SHALL BE HOOPS, AND SPACING SHALL NOT EXCEED d_l2.

WHEN WELDED SPLICES OR MECHANICAL CONNECTION ARE USED, NOT MORE THAN ALTERNATE BARS MAY BE SPLICED AT ANY SECTION WITH VERTICAL DISTANCE BETWEEN SPLICES 24 IN. (610 mm) OR MORE

CLEAR HEIGHT OF COL.

COL. TIES

S_t

6 d_b 3 IN. MIN.

6 d_b

d_b 90°

135°

SUPPLEMENTARY CROSSTIES
(ALTERNATE 90° AND 135° ENDS ON CONSECUTIVE CROSSTIES)

COL. HOOPS l_o

S_h

S_h = HOOP AND SUPPLEMENTARY CROSSTIE SPACING, NOT TO EXCEED $B_s/4$ OR 4 in. (102 mm)

S_o = COLUMN TIE SPACING, NOT TO EXCEED 8d_b OF VERTICALS, 24 d_b OF TIES, OR $B_s/2$ OR 12 IN. (305mm)

S_t = 2S_o

B_s = SMALLER DIMENSION OF COLUMN CROSS SECTION

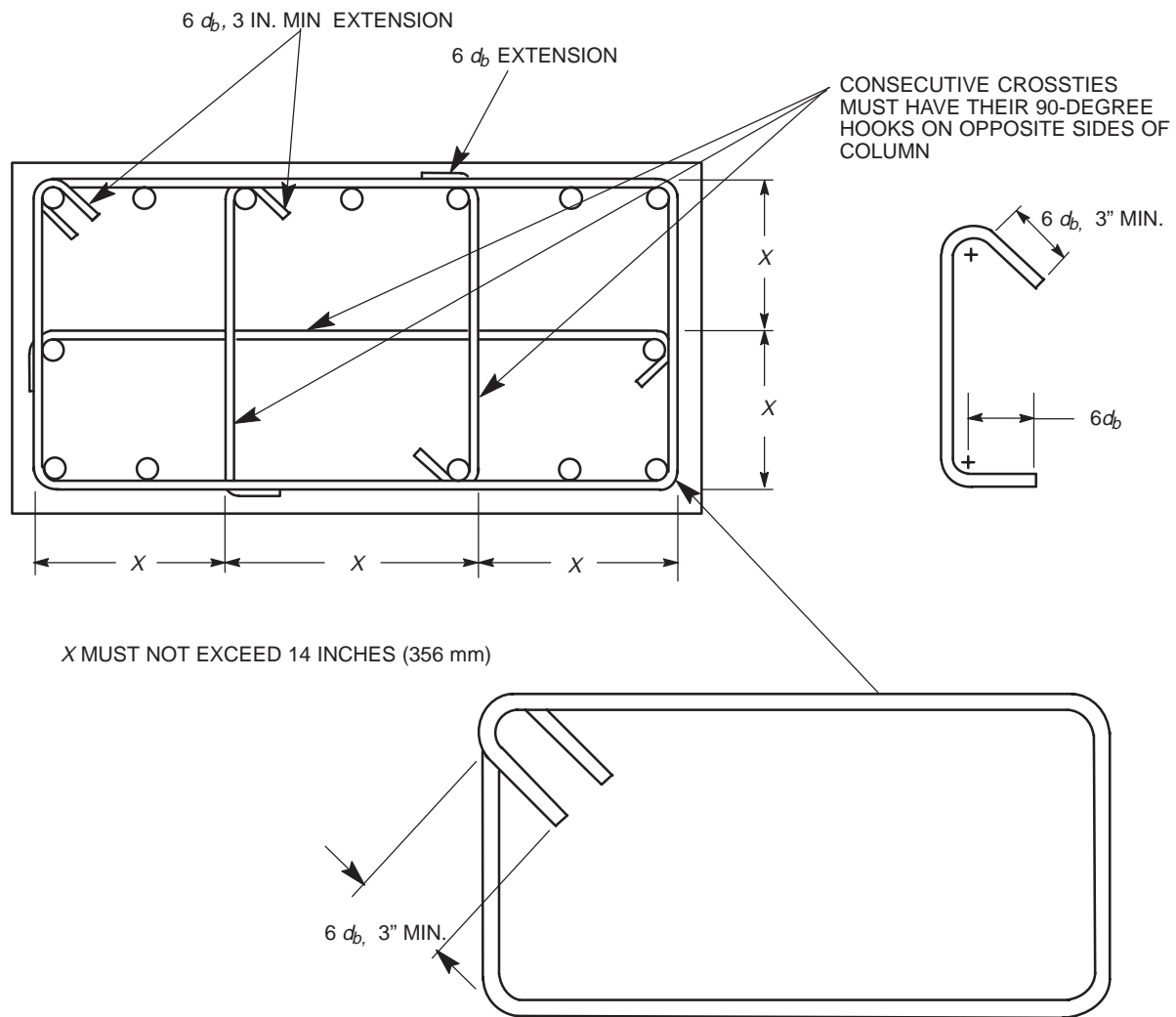l_o = LARGEST COLUMN DIMENSION, BUT NOT LESS THAN ONE-SIXTH CLEAR HEIGHT, OR 18 IN. (457 mm)

FIGURE VII-3
LONGITUDINAL AND TRANSVERSE REINFORCEMENT
FOR COLUMNS OF SMRFs

Reproduced from ACI Detailing Manual (1994)
Courtesy of American Concrete Institute

6 d_b, 3 IN. MIN EXTENSION

6 d_b EXTENSION

CONSECUTIVE CROSSTIES MUST HAVE THEIR 90-DEGREE HOOKS ON OPPOSITE SIDES OF COLUMN

6 d_b, 3" MIN.

6d_b

X

X

X

X

X

X MUST NOT EXCEED 14 INCHES (356 mm)

6 d_b, 3" MIN.

FIGURE VII-4
EXAMPLE OF TRANSVERSE REINFORCEMENT IN SMRF COLUMNS

Reproduced from *ACI Detailing Manual (1994)*
Courtesy of American Concrete Institute

VII CONCRETE

Section 1921.2.1.6 Precast Lateral-force-resisting Systems
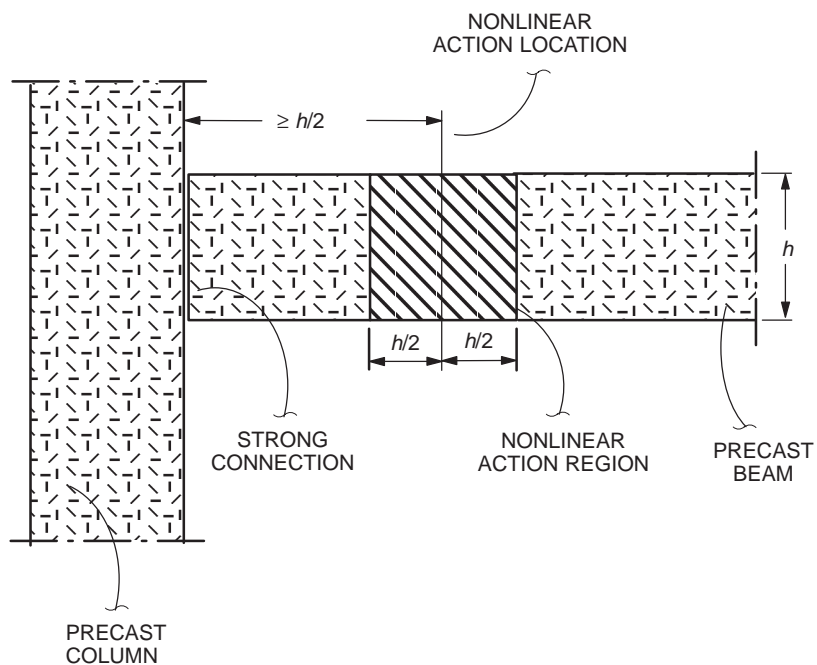
1921.1 – New Terminology

DRY CONNECTION

WET CONNECTION

STRONG CONNECTION

NONLINEAR ACTION REGION

NONLINEAR ACTION LOCATION

DETAIL 2.7.1
Monolithic Emulation
Jointed Precast

VII CONCRETE

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20. **Spacing of Lateral-force-resisting Systems.** In structures having precast gravity systems, the lateral-force-resisting system shall be one of the systems listed in Table 16-N and shall be well-distributed using one of the following methods: (Section 1921.2.1.7)

    a. The lateral-force-resisting systems 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. (See Detail 2.1.7a)

PRECAST
GRAVITY SYSTEM

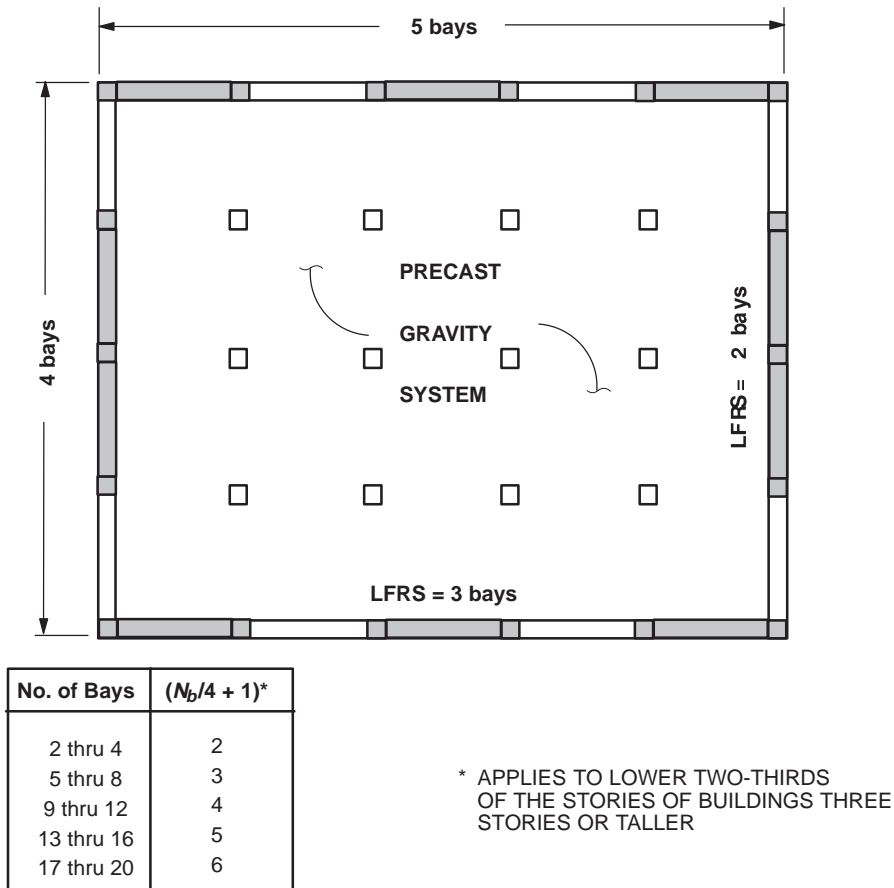LATERAL-FORCE-RESISTING
SYSTEMS (LFRS)

W

$L \le 3W$
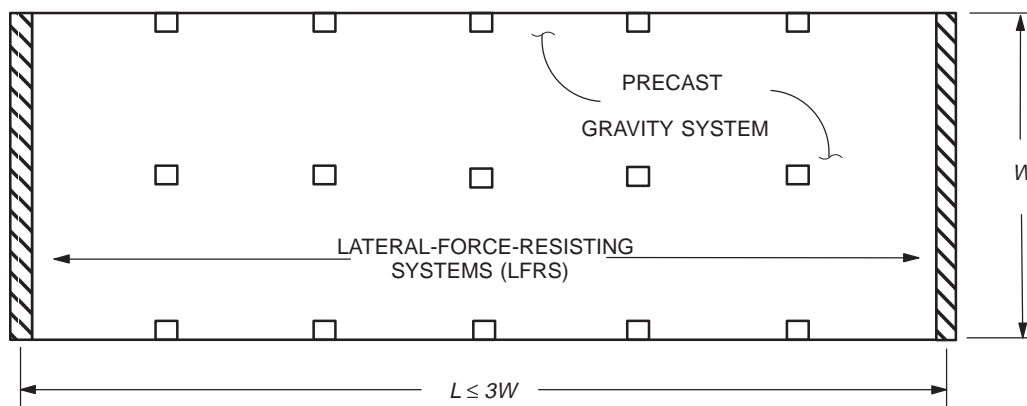
Span of diaphragm between LFRS not more than three times width of diaphragm

DETAIL 2.1.7a

**DISTRIBUTION OF LATERAL-FORCE-RESISTING SYSTEM IN
STRUCTURES HAVING PRECAST GRAVITY SYSTEMS**

b. 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. (See Detail 2.1.7b)

No. of Bays	$(N_b/4 + 1)^*$
2 thru 4	2
5 thru 8	3
9 thru 12	4
13 thru 16	5
17 thru 20	6

* APPLIES TO LOWER TWO-THIRDS
OF THE STORIES OF BUILDINGS THREE
STORIES OR TALLER

When LRFS consist of SMRF

DETAIL 2.1.7b

DISTRIBUTION OF LATERAL-FORCE-RESISTING SYSTEM IN STRUCTURES HAVING PRECAST GRAVITY SYSTEMS

21. **Beam to Column Connections.** All beam-to-column connections that are not part of the lateral-force-resisting system shall be designed in accordance with Section 1921.2.1.7.

22. **Deformation Compatibility.** Complete calculations for the deformation compatibility of the gravity load carrying system shall be made in accordance with Section 1633.2.4

using cracked section stiffnesses in the lateral-force-resisting system and the diaphragm. (Section 1921.2.1.7)

> ### SMRF REQUIREMENTS — JOINTS
> ### (SEISMIC ZONES 3 AND 4)

23. **Forces in Longitudinal Beam Reinforcement (Seismic Zones 3 and 4).** Forces in longitudinal beam reinforcement at the joint face should be determined by assuming that the stress in the flexural tensile reinforcement is 1.25 f_y. (Section 1921.5.1.1)

24. **Shear Strength of the Joint (Seismic Zones 3 and 4).** The nominal shear strength of the joint should not exceed the forces specified in Section 1921.5.3.

25. **Minimum Column Depth (Seismic Zones 3 and 4).** Where longitudinal beam reinforcing bars extend through a joint, the column dimension parallel to the beam reinforcement should not be less than 20 times the diameter of the largest longitudinal bar for normal-weight concrete. (Section 1921.5.1.4)

26. **Transverse Reinforcement (Seismic Zones 3 and 4).** The transverse reinforcement requirements of Section 1921.5.2 should be clearly detailed on the plans.

27. **Development Length for Reinforcement in Tension (Seismic Zones 3 and 4).** Plans should clearly indicate the development length of joint bars in tension in accordance with Section 1921.5.4. (See Figure VII-5.)
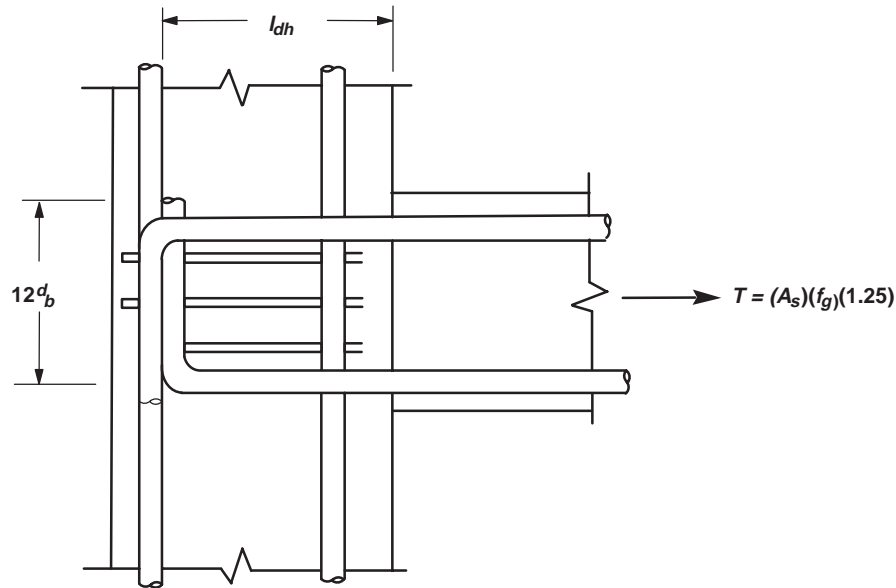
FIGURE VII- 5
DEVELOPMENT LENGTH FOR REINFORCEMENT IN TENSION

Development length, l_{dh}, for a bar with a standard 90-degree hook in normal-weight aggregate concrete should be $\geq 8d_b$, 6 inches (152 mm) and $f_y d_b / 65 \sqrt{f'_c}$. (Section 1921.5.4.1)

> **SHEAR WALLS, DIAPHRAGMS AND TRUSSES**
> **(SEISMIC ZONES 3 AND 4)**

28. **Reinforcement Requirements for Shear Walls (Seismic Zones 3 and 4).** The following requirements of Section 1921.6.2 should be met:

    a. The reinforcement ratio for shear walls should not be less than 0.0025 along the longitudinal and transverse axes.

    b. Reinforcement spacing each way should not exceed 18 inches (457 mm).

    c. At least two curtains of reinforcement should be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f'_c}$ (For **SI**: $0.166A_{cv}\sqrt{f'_c}$).

    d. When V_u in the plane of the wall exceeds $A_{cv}\sqrt{f'_c}$ (For **SI**: $0.08A_{cv}\sqrt{f'_c}$), horizontal reinforcement terminating at the edges of shear walls shall have a standard hook engaging the edge reinforcement, or the edge reinforcement shall be enclosed in "U" stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

e. Continuous reinforcement in shear walls should be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section 1921.5.4.

f. Splices in horizontal reinforcement should be staggered and splices in two curtains, where used, should not occur in the same location.

29. **Discontinuous Shear Walls (Seismic Zones 3 and 4).** Columns supporting discontinuous wall elements should be reinforced in accordance with Section 1921.4.4.5. (Section 1921.6.9)

30. **Reinforcement Requirements for Trusses, Diaphragms, Struts, Ties, Chords and Collectors (Seismic Zones 3 and 4).** The following reinforcement requirements of Section 1921.6.2.3 should be met:

a. Structural-truss members, struts, ties and collector members with compressive stresses exceeding $0.2f'_c$ should have special transverse reinforcement, as specified in Section 1921.4.4, over the total length of the member.

b. Continuous reinforcement in shear walls, diaphragms, trusses, struts, ties, chords and collector members should be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section 1921.5.4. Splices in horizontal reinforcement should be staggered. Splices in two curtains, where used, should not occur in the same location.

31. **Shear Strength of Shear Walls and Diaphragms (Seismic Zones 3 and 4).** The shear strength requirements for shear walls and diaphragms set forth in Section 1921.6.5 should be met.

32. **Effective Shear Wall Flange Width (Seismic Zones 3 and 4).** The effective flange widths to be used in the design of I-, L-, C- or T-shaped sections should not be assumed to extend further from the face of the web than (1) one-half the distance to an adjacent shear wall web, or (2) 15 percent of the total wall height for the flange in compression or 30 percent of the total wall height for the flange in tension, not to exceed the total projection of the flange. (Section 1921.6.6.2)

33. **Walls with $P_u > 0.35P_o$ (Seismic Zones 3 and 4).** Walls and portions of walls with $P_u > 0.35P_o$ should not be considered to contribute to the calculated strength of the structure for resisting earthquake-induced forces. Such walls should meet the deformation compatibility requirements of Section 1633.2.4. (Section 1921.6.6.3)

34. **Requirement for Shear Wall Boundary Zones (Seismic Zones 3 and 4).** Shear walls and portions of shear walls not meeting the conditions of Section 1921.6.6.4 should have boundary zones at each end in accordance with Section 1921.6.6.6.

35. **Coupling Beams (Seismic Zones 3 and 4).** Horizontal members with a clear span-to-effective-depth ratio of less than 4 and factored shear force exceeding $4\sqrt{f'_c}\,b_w d$ (For **SI:** $0.33\sqrt{f'_c}\,b_w d$) and which interconnect shear walls should be provided with special shear reinforcement in accordance with Section 1921.6.10.2.

36. **Limitation on Precast Floor Systems (Seismic Zones 3 and 4).** A cast-in-place topping on a precast floor system may serve as the diaphragm, provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces. Collector and boundary elements in topping slabs placed over precast floor and roof elements shall not be less than 3 inches or 6 times the diameter of the largest reinforcement in the topping slab. (Sections 1921.6.11 and 1921.6.12)

37. **Minimum Thickness of Diaphragms (Seismic Zones 3 and 4).** Diaphragms resisting earthquake loads should not be less than 2 inches (51 mm) thick. (Section 1921.6.12)

38. **Walls Piers Not a Part of Lateral-load-resisting System (Seismic Zones 3 and 4).** Wall piers not designed as a part of the SMRF should have transverse reinforcement designed to meet the requirements of Section 1921.6.13.2.

39. **Wall Segments with Horizontal Length-to-thickness Ratio < 2^1/$_2$ (Seismic Zones 3 and 4).** Wall segments with a horizontal length-to-thickness ratio less than 2^1/$_2$ should be designed as columns. (Section 1921.6.13.3)

**FRAME MEMBERS NOT PART OF THE
LATERAL-FORCE-RESISTING SYSTEM
IN SEISMIC ZONES 2, 3 AND 4**

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40. **Detailing Requirements.** Frame members assumed not to contribute to lateral resistance shall be detailed according to Section 1921.7.2 or 1921.7.3, depending on the magnitude of moments induced in those members when subjected to Δ_M. (Section 1921.7.1) (See Figures VII-1-a and VII-1-b, pages 30 and 31, for column tie spacing requirements)

    a. When the induced moments and shears under lateral displacements of Section 1921.7.1 combined with the factored gravity moments and shear loads do not exceed the design moment and shear strength of the frame member, the following conditions in Sections 1921.7.2.1, 1921.7.2.2 and 1921.7.2.3 shall be satisfied. For this purpose, the load combinations ($1.4D + 1.4L$) and $0.9D$ shall be used. (Section 1921.7.2)

    b. When the induced moments under lateral displacements of Section 1921.7.1 exceed the design moment strength of the frame member, or where induced moments are not calculated, the conditions in Section 1921.7.3.1 through 1921.7.3.3 shall be satisfied. (Section 1921.7.3)
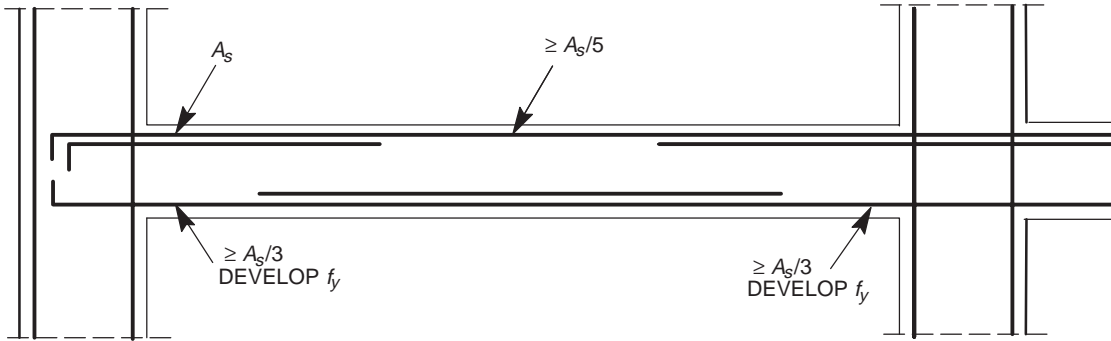
41. **Ties at Anchor Bolts.** Additional ties which engage at least four vertical column bars should be provided around anchor bolts which are set in the top of a column. Such ties should be within 5 inches (127 mm) of the top of the column and should consist of two No. 4 or three No. 3 bars. (Sections 1921.7.4 and 1921.4.4.8)

REQUIREMENTS FOR INTERMEDIATE MOMENT-RESISTING FRAMES IN SEISMIC ZONE 2

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42. **Beam Detailing Requirements (Seismic Zone 2).** Beams should be reinforced in accordance with Section 1921.8.4, as illustrated in Figure VII-6.

43. **Column Detailing Requirements (Seismic Zone 2).** Columns should be reinforced in accordance with Section 1921.8.5 as illustrated in Figure VII-6.

44. **Additional Ties around Anchor Bolts (Seismic Zone 2).** In addition to the column lateral ties required in Section 1907.1.3, additional ties which engage at least four vertical column bars should be provided around anchor bolts which are set in the top of a column. Such ties should be within 5 inches (127 mm) of the top of the column and should consist of two No. 4 or three No. 3 bars. (Sections 1921.8.5.5 and 1921.4.4.8)

Section 1921.8.4.1—LONGITUDINAL BARS

A_s

$\geq A_s/5$

$\geq A_s/3$
DEVELOP f_y

$\geq A_s/3$
DEVELOP f_y

WHERE S/2 DOES NOT
EXCEED $8d_b$, 24 d_{tie},
c_2, AND 12 IN. (305 mm)

COLUMN TIES

A_s (MIN.)

$h/6$, C, 18 IN. (457 mm)

S

$S/2$

$S/4$

h

$\leq$ 2 IN. (51 mm)

STIRRUPS
@ $d/2$

2 h
STIRRUPS
@ $d/4$

2 h STIRRUPS @ $d/4$,
BUT NOT MORE THAN
$8d_b$, 24d STIRRUP
OR 12 IN. (305 mm)
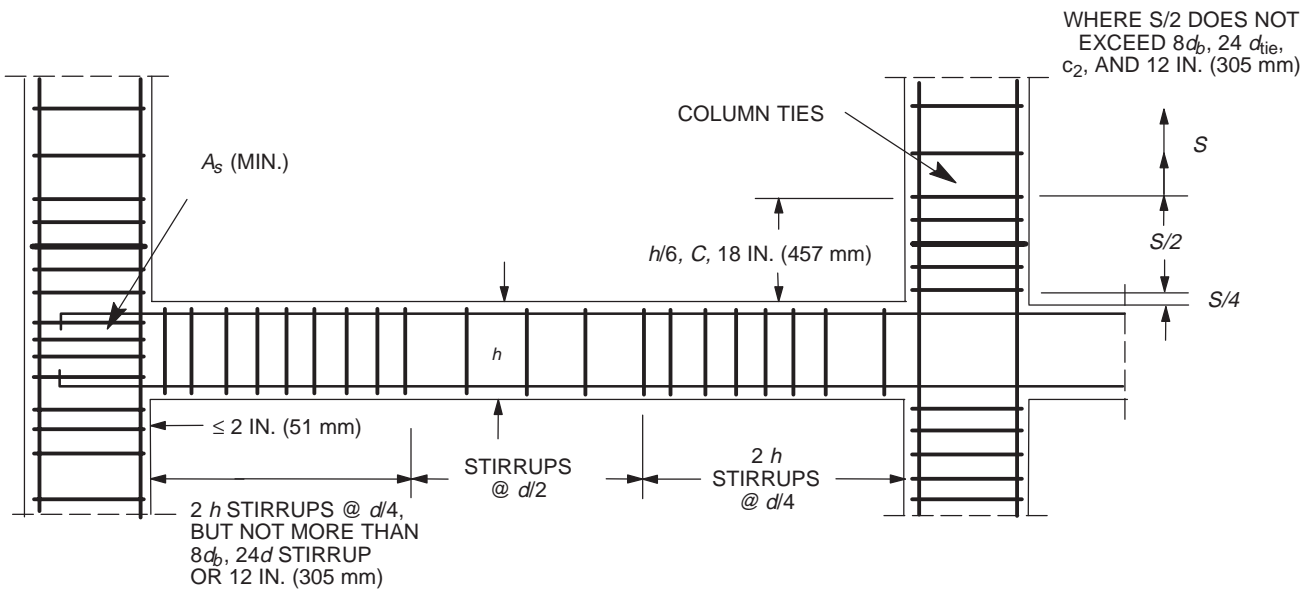
Section 1921.8.4.2—STIRRUPS

Section 1921.8.5—COLUMN TIES

FIGURE VII-6
LONGITUDINAL AND TRANSVERSE REINFORCEMENT
FOR SEISMIC ZONE 2 IMRFs

45. **Two-Way Slabs without Beams (Seismic Zone 2)**. Two-way slabs without beams should be reinforced in accordance with Section 1921.8.6 as illustrated in Figure VII-7.

SLAB COLUMN-STRIP

BANDED COLUMN-STRIP REINFORCEMENT
See also Section 1913.5.3.2

SLAB MIDDLE-STRIP
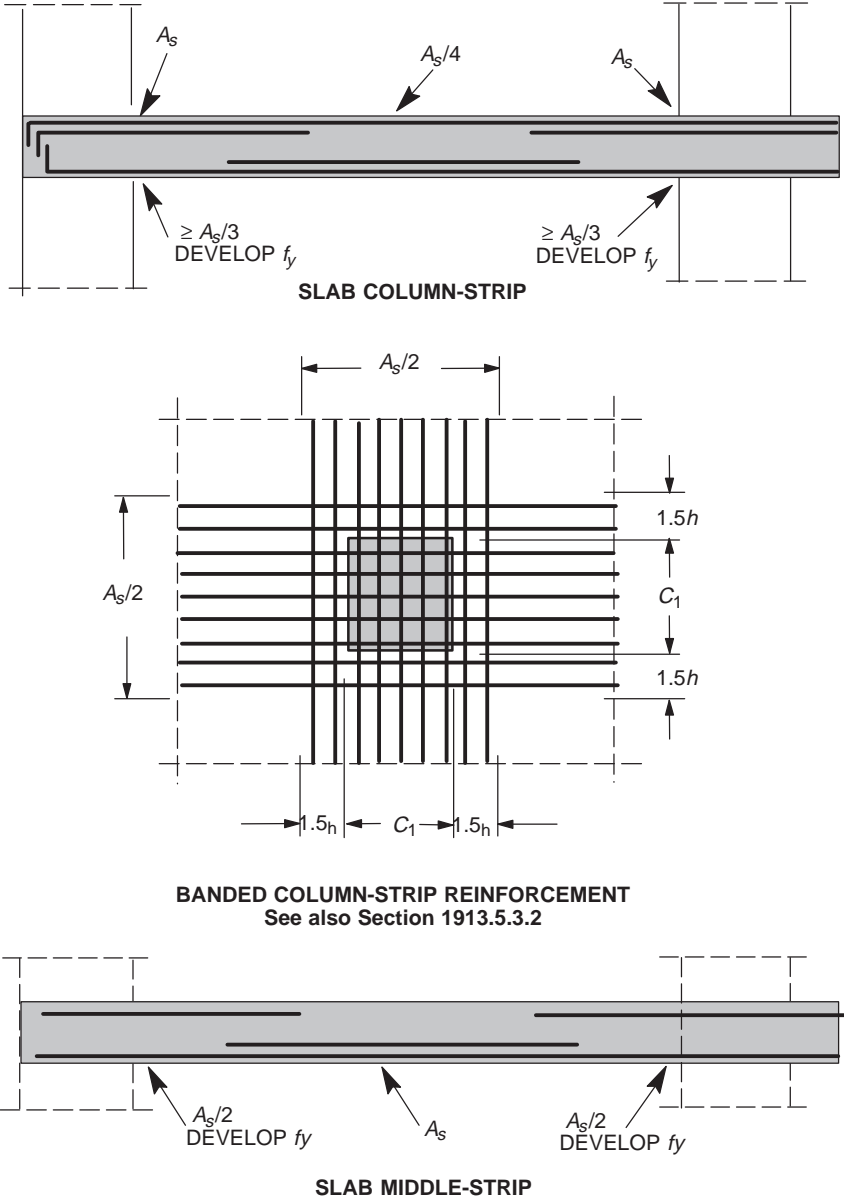
FIGURE VII-7
REINFORCEMENT REQUIREMENT FOR SEISMIC ZONE 2
TWO-WAY SLABS WITHOUT BEAMS (SECTION 1921.8.6)

VII CONCRETE

VIII MASONRY

VIII. MASONRY

A. Material Requirements

1. **Information Included with Masonry Specifications.** Masonry specifications should be provided on the plans. The following information should be provided:

    a. Mortar ingredients and proportions. (Sections 2102.2 and 2103.3)

    b. Grout ingredients and proportions, or minimum compressive strength. (Sections 2102.2 and 2103.4)

    c. Reinforcement type and strength. (Section 2102.2)

    d. Masonry unit description and strength. (Section 2102.2)

2. **Prohibited Materials in Seismic Zone 2.** The following materials should not be used as part of the vertical- or lateral-load-resisting systems (Section 2106.1.12.3):

    a. Type O mortar.

    b. Masonry cement.

    c. Plastic cement.

    d. Nonload-bearing masonry units.

    e. Glass block.

3. **Prohibited Materials in Seismic Zones 3 and 4.** The following materials should not be used as part of the vertical- or lateral-load-resisting systems (Section 2106.1.12.3):

    a. Types O and N mortar.

    b. Masonry cement.

    c. Plastic cement.

    d. Nonload-bearing masonry units, and

    e. Glass block.

4. **Verification of Masonry Compressive Strength.** Plans should specify the method by which the specified compressive strength of masonry f'_m will be verified: (1) masonry prism testing, (2) masonry prism test record, (3) unit strength method, or (4) testing prisms from constructed masonry. (Section 2105.3) Method description should include the following information:

a. Whether full or half allowable stresses were used in the design.

b. Number of, location, and type of tests to be taken, and at what time during construction.

c. Special inspection requirements.

d. Records and letters of certification required.

B. Working Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry

1. **Effective Area.** The effective area of masonry should be based on the minimum area as described in Section 2106.2.5.

2. **Distribution of Concentrated Vertical Loads in Walls.** The length of wall considered capable of working at the maximum allowable compressive stress to resist vertical concentrated loads should not exceed the center-to-center distance between loads or the width of bearing area plus four times the wall thickness. (Section 2106.2.7)

3. **Maximum Deflection of Elements Supporting Masonry.** It should be shown by calculation that elements supporting masonry do not exceed a vertical deflection of $^1/_{600}$ of the clear span. (Section 2106.2.9)

4. **Minimum Bearing Length for Lintels.** Plans should specify that the minimum bearing length for lintels supported by masonry is 4 inches (102 mm). (Section 2106.2.9)

5. **Modulus of Elasticity.** The modulus of elasticity for masonry (steel) should be determined in accordance with Section 2106.2.12. Note that the modulus of elasticity is NOT to be reduced by one-half when half allowable stresses are used in the design.

6. **Anchor Bolt Specifications.** Anchor bolt specifications should be provided on the plans. Specifications should be in accordance with the descriptions in Section 2106.2.14.1.

7. **Required Grout around Bolt.** Plans should indicate that embedded bolts will be grouted in place with at least 1 inch (25 mm) of grout between the bolt and masonry. (Section 2106.2.14.1)

8. **Anchor Bolt Detailing Requirements.** Minimum edge distances, embedment depths and spacing between bolts should be detailed on the plans in accordance with Sections 2106.2.14.2, 2106.2.14.3 and 2106.2.14.4.

9. **Load Combinations Applied to Design Forces.**

   a. Where strength design is used, the basic load combinations shall comply with Section 1612.2. Factored load combinations shall be multiplied by 1.1 where load combinations include seismic forces.

   b. Where allowable stress design is used, the basic load combinations shall comply with Section 1612.3.

C. **Reinforcement Requirements for Working Stress Design and Strength Design**

   1. **Minimum Distance between Parallel Bars.** Plans should specify that the minimum clear distance between parallel bars, except in columns, should not be less than the nominal diameter of the bars or 1 inch (25 mm). (Section 2106.3.3)

   2. **Anchorage of Flexural Reinforcement.** Flexural reinforcement should be anchored in accordance with Section 2106.3.4.

   3. **Anchorage of Shear Reinforcement.** Shear reinforcement should be anchored in accordance with Section 2106.3.5.

   4. **Lateral Ties.** Longitudinal bars for columns should be enclosed by lateral ties in accordance with Figure VIII-1 and Table VIII-1. (Section 2106.3.6)
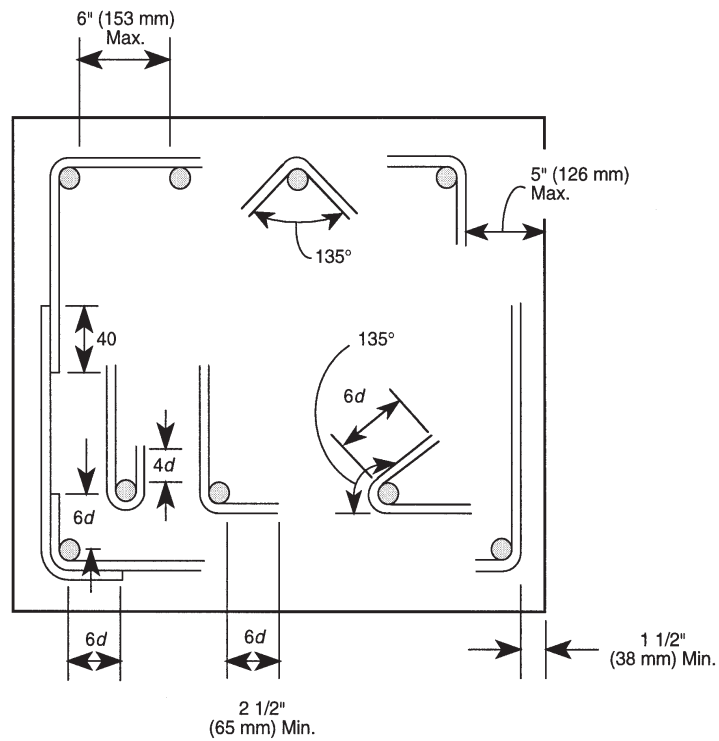
FIGURE VIII-1
LATERAL TIES

Reproduced from *Commentary to Chapter 21, Masonry of the Uniform Building Code*
Courtesy of The Masonry Society

TABLE VIII-1
MAXIMUM TIE SPACING BASED ON LONGITUDINAL BAR SIZE[a]

Longitudinal Bar Size		Maximum Tie Spacing
English Unit	**Metric Unit**	**Inches (mm)**
# 3	# 10	6 (152 mm)
# 4	# 13	8 (203 mm)
# 5	# 16	10 (254 mm)
# 6	# 19	12 (305 mm)
# 7	# 22	14 (356 mm)
# 8	# 25	16 (406 mm)
# 9	# 29	18 (457 mm)
#10	# 32	18 (457 mm)
# 11	# 36	18 (457 mm)

MAXIMUM TIE SPACING BASED ON TIE SIZE[1]

Tie Bar Size		Maximum Tie Spacing
English Unit	**Metric Unit**	**Inches (mm)**
# 3	# 10	18 (457 mm)
# 4	# 13	18 (457 mm)
# 5	# 16	18 (457 mm)

[1] Maximum tie spacing not to exceed the least column dimension.

5. **Column Anchor Bolt Ties.** Additional ties should be provided around anchor bolts which are set in the top of columns, in accordance with Section 2106.3.7.

D. General Requirements for Working Stress Design

1. **Special Inspection.** Special inspection should be specified on the plans for masonry construction or allowable design stresses specified in Section 2107 should be reduced by one-half. (Section 2107.1.2)

2. **Minimum Bearing Wall Thickness in Seismic Zones 3 and 4.** The nominal thickness of reinforced masonry bearing walls should not be less than 6 inches (152 mm) except as noted in Section 2107.1.3.1.

3. **Minimum Column Dimensions in Seismic Zones 3 and 4.** The least nominal dimension of a reinforced masonry column should be 12 inches (305 mm) except, if the allowable stresses are reduced by one-half, the minimum nominal dimension should be 8 inches (203 mm). (Section 2107.1.3.2)

4. **Design Loads for Shear Walls in Seismic Zones 3 and 4.** Design shear stresses in shear walls should be increased by a factor of 1.5. (Section 2107.1.7)

E.   Requirements for Working Stress Design—Reinforced Masonry

1. **Maximum Bar Size.** Bar sizes greater than No. 11 should not be used. (Section 2107.2.2.1)

2. **Maximum Steel Area.** Maximum steel area should be limited to 6 percent of the cell area without splices and 12 percent of the cell area with splices. (Section 2107.2.2.1)

3. **Minimum Development Length.** Minimum development lengths for beams, columns and walls should be specified on the plans in accordance with Section 2107.2.2.3.

4. **Minimum Lap Splice.** In no case should the lapped splice length be less than 30 bar diameters for compression and 40 bar diameters for tension. (Section 2107.2.2.6)

5. **Lap Splice Increases.** In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tensile stress F_s, the lap length of splices should be increased not less than 50 percent of the minimum required length. (Section 2107.2.12)

6. **Minimum Column Longitudinal Reinforcement.** The area of vertical reinforcement should not be less than 0.005 A_e and not more than 0.04 A_e. At least four No. 3 bars should be provided. The minimum clear distance between parallel bars in columns should be two and one-half times the bar diameter. (Section 2107.2.13.1)

7. **$h'_t > 30$.** Walls with a h'_t ratio greater than 30 should be based on an analysis taking into account the effects itemized in Section 2107.2.14.2.

F.   Allowable Stresses for Working Stress Design — Reinforced Masonry

1. **Maximum Reinforcement Bond Stress.** Bond stress in reinforcing bars should not exceed the allowable stresses set forth in Section 2107.2.2.4.

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WALLS
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2. **Allowable Stresses in Walls.** The following stresses should be shown not to be exceeded for walls:

a.   **Axial Compressive Stress** (Section 2107.2.5):

$$F_a = 0.25f'_m \left[1 - \left(\frac{h'}{140r}\right)^2\right] \quad \text{for } h'/r \leq 99 \qquad \text{Formula (7-11)}$$

$$F_a = 0.25f'_m \left(\frac{70r}{h'}\right)^2 \quad \text{for } h'/r > 99 \qquad \text{Formula (7-12)}$$

b.   **Flexural Compressive Stress** (Section 2107.2.6):

$$F_b = 0.33 \, f'_m, \quad 2{,}000 \text{ psi (13.8 MPa) maximum} \qquad \text{Formula (7-15)}$$

c.   **Shear Stress** in shear walls should be in accordance with Section 2107.2.9.

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COLUMNS

</div>

3.   **Allowable Stresses for Columns.** The following stresses should be shown not to be exceeded for columns:

a.   **Axial Compressive Stress** (Section 2107.2.5):

$$P_a = [0.25f'_m A_e + 0.65A_s F_{sc}]\left[1 - \left(\frac{h'}{140r}\right)^2\right] \quad \text{for } h'/r \leq 99 \quad \text{Formula (7-13)}$$

$$P_a = [0.25f'_m A_e + 0.65A_s F_{sc}]\left(\frac{70r}{h'}\right)^2 \quad \text{for } h'/r > 99 \qquad \text{Formula (7-14)}$$

where $F_a = P_a / A_e$

b.   **Bearing Stress** (Section 2107.2.10):

On full area,

$$F_{br} = 0.26 \, f'_m \qquad \text{Formula (7-23)}$$

On one-third area or less,

$$F_{br} = 0.38 \, f'_m \qquad \text{Formula (7-24)}$$

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FLEXURAL MEMBERS

</div>

4.   **Allowable Stresses in Flexural Members.** The following stresses should be shown not to be exceeded for flexural members:

a. **Compressive Stress** (Section 2107.2.6):

$$F_b = 0.33 \ f'_m, \ 2{,}000 \text{ psi (13.8 MPa) maximum} \qquad \text{Formula (7-15)}$$

b. **Shear Stress** (Section 2107.2.8):

No shear reinforcement,

$$F_v = 1.0 \sqrt{f'_m}, \ 50 \text{ psi maximum} \qquad \text{Formula (7-17)}$$

For **SI:** $F_v = 0.083 \sqrt{f'_m}$, 345 kPa maximum

Shear reinforcement designed to take entire shear force,

$$F_v = 3.0 \ \sqrt{f'_m}, \ 150 \text{ psi maximum} \qquad \text{Formula (7-18)}$$

For **SI:** $F_v = 0.25 \sqrt{f'_m}$, 1.0 MPa maximum

c. **Bearing Stress** (Section 2107.2.10):

On full area,

$$F_{br} = 0.26 \ f'_m \qquad \text{Formula (7-23)}$$

On one-third area or less,

$$F_{br} = 0.38 \ f'_m \qquad \text{Formula (7-24)}$$

5. **Combined Axial and Flexural Stresses.** Combined axial and flexural stresses should be investigated in accordance with Section 2107.2.7.

6. **Allowable Stress in Reinforcement.** Stresses in reinforcement should be shown not to exceed the allowable stresses set forth in Section 2107.2.11.

G. Allowable Stresses for Working Stress Design—Unreinforced Masonry

WALLS

1. **Allowable Stresses for Walls.** The following stresses should be shown not to be exceeded for walls:

a. **Axial Compressive Stress** (Section 2107.3.2):

$$F_a = 0.25 \, f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad \text{for} \; h'/r \leq 99 \qquad \text{Formula (7-39)}$$

$$F_a = 0.25 \, f'_m \left(\frac{70r}{h'} \right)^2 \quad \text{for} \; h'/r > 99 \qquad \text{Formula (7-40)}$$

b. **Flexural Compressive Stress** (Section 2107.3.3):

$$F_b = 0.33 \, f'_m, \; 2,000 \; \text{psi (13.8 MPa) maximum}$$

c. **Tensile Stress** should be in accordance with Section 2107.3.5.

d. **Shear Stress** in shear walls should be in accordance with Section 2107.3.7.

COLUMNS

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2. **Allowable Stresses for Columns.** The following stresses should be shown not to be exceeded for columns:

a. **Axial Compressive Stress** (Section 2107.3.2):

$$F_a = 0.25 \, f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad \text{for} \; h'/r \leq 99 \qquad \text{Formula (7-39)}$$

$$F_a = 0.25 \, f'_m \left(\frac{70r}{h'} \right)^2 \quad \text{for} \; h'/r > 99 \qquad \text{Formula (7-40)}$$

b. **Bearing Stress** (Section 2107.3.8):

On full area,

$$F_{br} = 0.26 \, f'_m \qquad \text{Formula (7-45)}$$

On one-third area or less,

$$F_{br} = 0.38 \, f'_m \qquad \text{Formula (7-46)}$$

FLEXURAL MEMBERS

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3. **Allowable Stresses for Flexural Members.** The following stresses should be shown not to be exceeded for flexural members:

a. **Compressive Stress** (Section 2107.3.3):

$$F_b = 0.33f'_m, 2{,}000 \text{ psi } (13.8 \text{ MP}_a) \text{ maximum} \qquad \text{Formula (7-41)}$$

b. **Shear Stress** (Section 2107.3.6):

$$F_v = 1.0 \sqrt{f'_m}, 50 \text{ psi maximum} \qquad \text{Formula (7-43)}$$

For **SI:** $F_v = 0.083 \sqrt{f'_m}$, 345 kPa maximum

c. **Bearing Stress** (Section 2107.3.8):

On full area,

$$F_{br} = 0.26 \ f'_m \qquad \text{Formula (7-45)}$$

On one-third area or less,

$$F_{br} = 0.38 \ f'_m \qquad \text{Formula (7-46)}$$

COMBINED AXIAL AND FLEXURAL STRESSES

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4. **Combined Axial and Flexural Stresses.** Combined axial and flexural stresses should be investigated in accordance with Section 2107.3.4.

H. General Requirements for Strength Design

1. **Special Inspection.** Special inspection during construction should be provided as set forth in Section 1701.5, Item 7. (Section 2108.1.2)

2. **Anchor Bolt Placement.** Anchor bolts should be placed so as to meet the edge distance, embedment depth and spacing requirements of Sections 2106.2.14.2, 2106.2.14.3 and 2106.2.14.4.

I. Requirements for Strength Design—Reinforced Masonry

1. **Maximum Reinforcement.** Bar sizes greater than No. 9 should not be used. The diameter of a bar should not exceed one-fourth the least dimension of a cell and no more than two bars should be placed in a cell of a wall or a wall frame. (Section 2108.2.2.1)

2. **Clear Distance between Vertical Reinforcing Bars in Columns and Piers.** In columns and piers, the clear distance between vertical reinforcing bars should not be
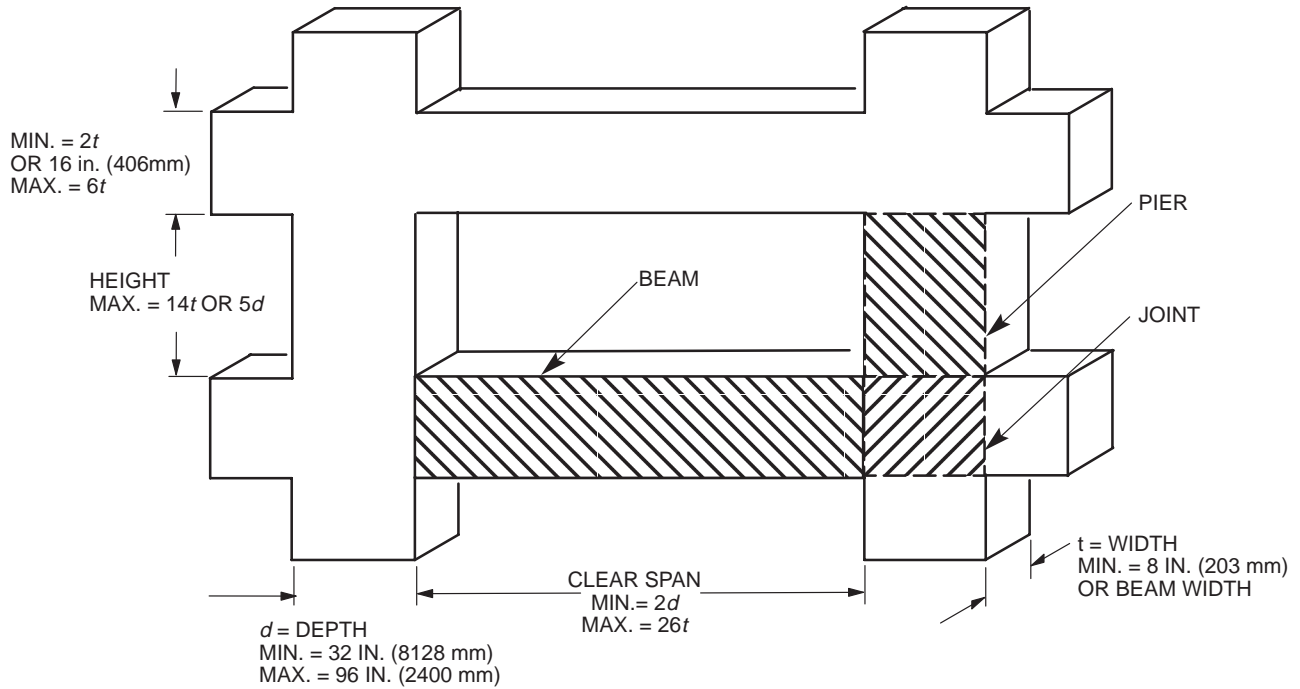
less than one and one-half times the nominal bar diameter, or less than $1^1/_2$ inches (38 mm). (Section 2108.2.2.2)
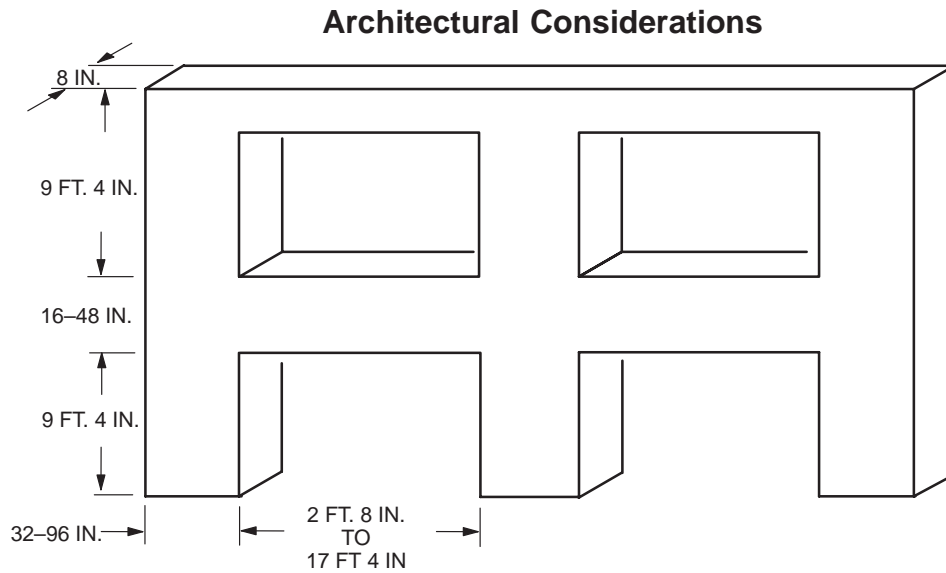
3. **Development Length.** Plans should specify development lengths in accordance with Section 2108.2.2.6. The minimum embedment length of reinforcement should be 12 inches (305 mm).

4. **Minimum Lap Splice Lengths.** Plans should specify minimum lap splice lengths in accordance with Section 2108.2.2.7.

5. **Design of Beams, Piers and Columns.** The following comments pertain to the design requirements for beams, piers and columns as set forth in Section 2108.2.3:

   a. **Limitations on f'_m.** The value of f'_m should not be less than 1,500 psi (10.3 MPa) and, for computational purposes, not more than 4,000 psi (27.6 MPa). (Section 2108.2.3.1)

   b. **Reinforcement.** Reinforcement should be detailed on the plans in accordance with the general requirements set forth in Section 2108.2.3.7, beam requirements set forth in Section 2108.2.3.10, pier requirements set forth in Section 2108.2.3.11, and column requirements set forth in Section 2108.2.3.12.

   c. **Dimensional Limits.** Beams, piers and columns should meet the dimensional requirements of Section 2108.2.3.9.

6. **Maximum Reinforcement for Walls Loaded Out-of-plane.** The maximum reinforcement ratio for walls loaded out-of-plane should not exceed $0.5\rho_b$. (Section 2108.2.4.2)

7. **Wall Design for In-plane Loads.** The following comments pertain to the design requirements for walls loaded in-plane as set forth in Section 2108.2.5:

   a. **Limitations on f'_m.** The value of f'_m should not be less than 1,500 psi (10.3 MPa) nor greater than 4,000 psi (27.6 MPa). (Section 2108.2.5.1)

   b. **Reinforcement.** Reinforcement should be detailed on the plans in accordance with the requirements set forth in Section 2108.2.5.2.

   c. **Boundary Members.** Boundary members should be provided in accordance with Section 2108.2.5.6.

8. **Design of Moment-resisting Wall Frames.** The following comments pertain to the design requirements for moment-resisting wall frames as set forth in Section 2108.2.6:

   a. **Dimensional Limits.** Beams and piers should meet the dimensional requirements of Section 2108.2.6.1.2. (See Figures V111-2 and V111-3.)

MIN. = 2t
OR 16 in. (406mm)
MAX. = 6t

HEIGHT
MAX. = 14t OR 5d

BEAM

PIER

JOINT

t = WIDTH
MIN. = 8 IN. (203 mm)
OR BEAM WIDTH

CLEAR SPAN
MIN.= 2d
MAX. = 26t

d = DEPTH
MIN. = 32 IN. (8128 mm)
MAX. = 96 IN. (2400 mm)

FIGURE VIII-2
DIMENSIONAL LIMITS FOR MOMENT-RESISTING WALL FRAMES (SECTION 2108.2.6.1.2)

Architectural Considerations

8 IN.

9 FT. 4 IN.

16–48 IN.

9 FT. 4 IN.

32–96 IN.

2 FT. 8 IN.
TO
17 FT 4 IN

Dimensional Limits for 8-inch Units

12 IN.

14 FT. 0 IN.

24–72 IN.

14 FT. 0 IN.

32–96 IN.

4–26 FT.
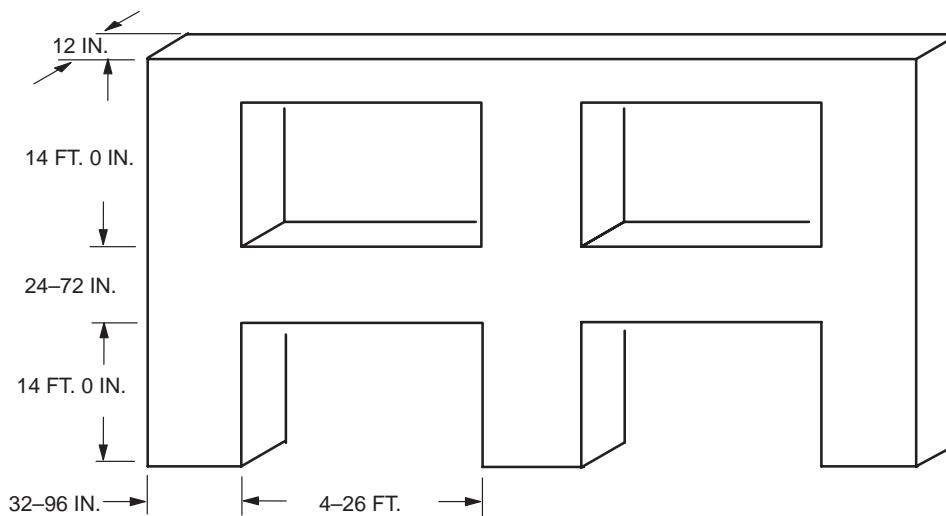
For **SI:** 1 inch = 25.4 mm; 1 foot = 304.8 mm.

Dimensional Limits for 12-inch Units

**FIGURE VIII-3
ARCHITECTURAL CONSIDERATIONS FOR MOMENT-RESISTING WALL FRAMES
(SECTION 2108.2.6.1.2)**

b. **Limitations on f'_m.** The value of f'_m should not be less than 1,500 psi (10.3 MPa) or more than 4,000 psi (27.6 MPa). (Section 2108.2.6.2.3)

c. **Reinforcement.** Reinforcement should be detailed on the plans in accordance with the requirements set forth in Sections 2108.2.6.2.4, 2108.2.6.2.5, 2108.2.6.2.6 and 2018.2.6.2.9.

J.   Empirical Design of Masonry

1. **Limited Application for Empirical Design.** Empirical masonry design should not be used unless the structure is located in Seismic Zone 0 or 1, the basic wind speed is less than 80 miles per hour (129 km/h) and its use is approved by the building official. (Section 2109)

2. **Empirical Design Requirements.** Empirical masonry design should comply with the requirements of Section 2109.
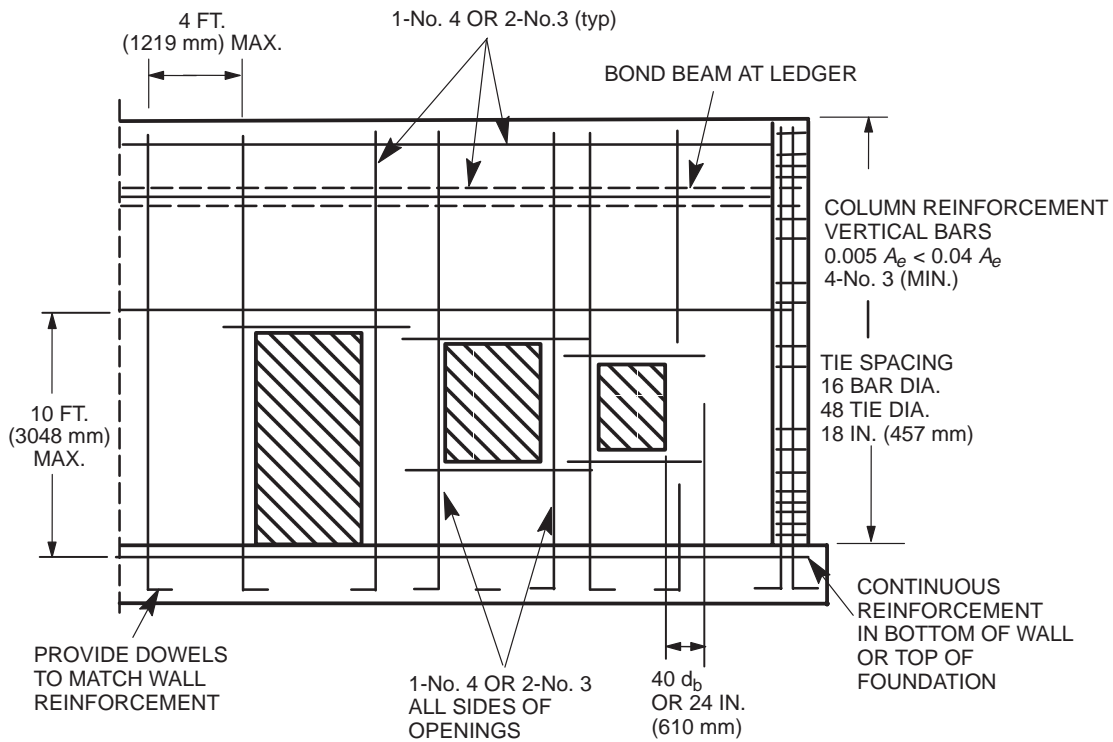
K.   Glass Masonry

1. **Limited Use of Glass Block.** Glass block should not be used except in nonload-bearing exterior or interior walls and in openings which might otherwise be filled with windows. (Section 2110.1)

2. **Glass Block Requirements.** Glass block should comply with the requirements of Section 2110.

L.   Seismic Design Requirements

1. **Wall and Column Reinforcement in Seismic Zone 2.** Wall and column reinforcement should comply with the requirements shown in Figure VIII-4. (Section 2106.1.12.3)

2. **Wall and Column Reinforcement in Seismic Zones 3 and 4.** Wall and column reinforcement should comply with the requirements shown in Figure VIII-5. (Section 2106.1.12.4)

3. **Maximum Shear Wall Reinforcement in Seismic Zones 3 and 4.** The maximum spacing of shear wall reinforcement in each direction should not exceed one half the length of the element, one half the height of the element or 48 inches (1219 mm). (Section 2106.1.12.4, Item 2.1)

4 FT.
(1219 mm) MAX.

1-No. 4 OR 2-No.3 (typ)

BOND BEAM AT LEDGER

COLUMN REINFORCEMENT
VERTICAL BARS
$0.005\ A_e < 0.04\ A_e$
4-No. 3 (MIN.)

TIE SPACING
16 BAR DIA.
48 TIE DIA.
18 IN. (457 mm)

10 FT.
(3048 mm)
MAX.

CONTINUOUS
REINFORCEMENT
IN BOTTOM OF WALL
OR TOP OF
FOUNDATION

PROVIDE DOWELS
TO MATCH WALL
REINFORCEMENT

1-No. 4 OR 2-No. 3
ALL SIDES OF
OPENINGS

$40\ d_b$
OR 24 IN.
(610 mm)

FIGURE VIII-4
WALL AND COLUMN REINFORCEMENT REQUIREMENTS FOR
SEISMIC ZONE 2 (SECTION 2106.1.12.3)

MINIMUM REIN. (EACH DIRECTION)
= 0.0007 x WALL AREA
(SUM ≥ 0.0002 x WALL AREA)

COLUMN TIES*

4 FT.
(1219 mm) MAX.

BOND BEAM AT LEDGER

8 IN. (203 mm) O.C
[(h_c/6 IN. (152 mm) OR
18 IN. (457 mm)]

16 BAR DIA.
28 TIE DIA.
18 IN. (457 mm)

8 IN. (203 mm) O.C
[(h_c/6 IN. (152 mm) OR
18 In. (457 mm)]

PROVIDE DOWELS
TO MATCH WALL
REINFORCEMENT

1-No. 4 OR 2-No. 3
ALL SIDES OF
OPENINGS

40 d_b OR 24 IN.
(610 mm)

*8 IN. (203 mm)
O.C. ENTIRE
HEIGHT IF EQ
OVERTURNING
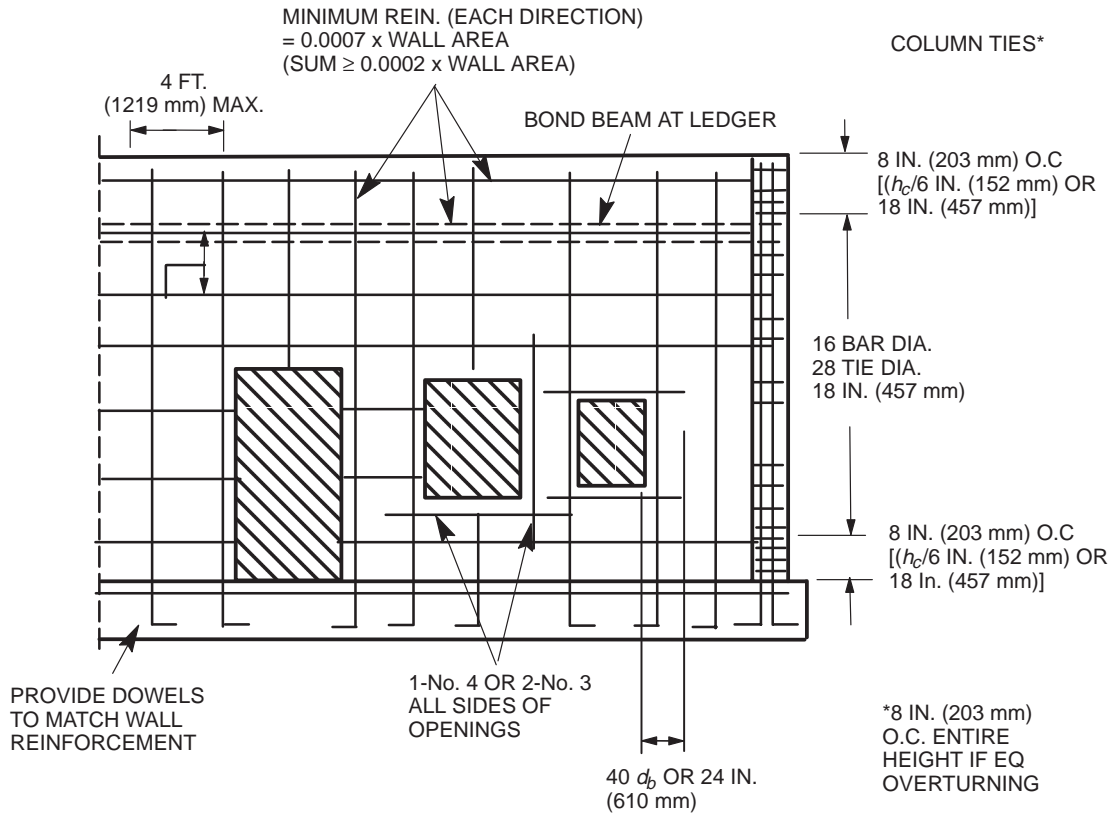
FIGURE VIII-5
WALL AND COLUMN REINFORCEMENT REQUIREMENTS FOR
SEISMIC ZONES 3 AND 4 (SECTION 2106.1.12.4)

IX   STEEL

IX. STEEL

A.   **Cold-formed Steel Construction.** The design of cold-formed steel structural members should be in accordance with Chapter 22, Division VI or VII. (Section 2205.4)

B.   **Cold-formed Stainless Steel Construction.** The design of cold-formed stainless steel structural members should be in accordance with approved national standards. See Section 2202.2. (Section 2205.5)

C.   **Open-web Steel Joists and Joist Girders.** The design, manufacture and use of steel joist, K, LH and KLH series and joist girders should be in accordance with Chapter 22, Division IX. (Section 2205.7)

D.   **Steel Storage Racks.** Steel storage racks should be designed either in accordance with the provisions of Chapter 22, Division X, or in accordance with the nonstructural component provisions or the nonbuilding structure provisions of Chapter 16. (Section 2205.8, Tables 16-O and 16-P)

E.   **Steel Cables.** Structural applications of steel cables for buildings should be in accordance with the provisions of Chapter 22, Division XI. (Section 2205.9)

F.   **Welding.** Welding procedures, welder qualification requirements and welding electrodes should be in accordance with Chapter 22, Division II, III, VI or VII, and approved national standards. (Section 2205.10)

G.   **Bolts.** The use of high-strength A 325 and A 490 bolts should be in accordance with the requirements of Chapter 22, Division II and III. (Section 2205.11)

H.   **Allowable Stress Design Requirements.** (Note: Section references are from Chapter 22, Division III.)

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GENERAL

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1.   **American Society for Testing and Materials (ASTM) Designations.** One of the following ASTM designations should be specified for the structural steel: A 36, A 53 (Grade B), A 242, A 441, A 500, A 501, A 514, A 529, A 570 (Grades 40, 45 and 50), A 572, A 588, A 606, A 607, A 618, A 709 or A 852. [Section 2208, AISC-ASD (1989), Chapter A, Item 1a]

2. **Steel Bolts.** Steel bolts should be specified with one of the following standard specifications: A 307, A 325, A 449, A 490, A 563 or F 436. [Section 2208, AISC-ASD (1989), Chapter A, Item 4]

3. **Anchor Bolts and Threaded Rods.** Anchor bolts and threaded rod steel should be specified with one of the following standard specifications: A 36, A 194 Grade 7, A 354, A 449, A 588 or A 687. [Section 2208, AISC-ASD (1989), Chapter A, Item 5]

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TENSION MEMBER DESIGN

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4. **Allowable Stress.** The allowable stress F_t should not exceed $0.60F_y$ on the gross area or $0.50F_u$ on the effective net area. [Section 2208, AISC-ASD (1989), Chapter D1]

5. **Gross and Net Areas.** The gross and net areas of members should be determined in accordance with AISC-ASD (1989) Chapters, B1 and B2, respectively.

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COMPRESSION MEMBER DESIGN

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6. **Effective-length Factor K.** The effective-length factor K should be determined in accordance with Section 2208, AISC-ASD (1989), Chapters E1 and C2.

7. **Allowable Stress.** The allowable stress in compression members should not exceed the value determined in AISC-ASD (1989), Chapter E2.

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FLEXURAL MEMBER DESIGN

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8. **Members with Compact Sections with Unbraced Length Less than L_c.** The allowable stress for members with compact sections with an unbraced length less than L_c should not exceed $0.66F_y$. [Section 2208, AISC-ASD (1989), Chapter F1, Item 1]

9. **Members with Unbraced Length Greater than L_c.** The allowable bending stress in tension should not exceed $0.60F_y$. The allowable bending stress in compression should not exceed the larger value from Equations (F1-6) or (F1-7) and (F1-8). [AISC-ASD (1989), Chapter F1, Item 3]

10. **Allowable Shear Stress.** For $h/t_w \leq 380/\sqrt{F_y}$ (For **SI:** $h/t_w \leq 2.24\sqrt{E/F_y}$) on the overall depth times the web thickness, the allowable shear stress should not exceed $F_v = 0.40F_y$. For $h/t_w \geq 380/\sqrt{F_y}$ (For **SI:** $h/t_w \geq 2.24\sqrt{E/F_y}$) the

allowable shear stress is on the clear distance between flanges times the web thickness and should not exceed $F_v = \dfrac{F_y}{2.89}(C_v) \leq 0.40F_y$. [AISC-ASD (1989), Chapter F4]

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COMPOSITE CONSTRUCTION DESIGN

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11. **Effective Width.** The portion of the effective width of the concrete slab on each side of the beam centerline should not exceed [Section 2208, AISC-ASD (1989) Chapter A, Item #1]:

    a. $^1/_8$ of the beam span, center-to-center of supports;

    b. $^1/_2$ the distance to the centerline of the adjacent beam; or

    c.   the distance from the beam centerline to the edge of the slab.

12. **Loading Conditions for Encased Beams.** Encased beams should be designed for the various loading conditions set forth in AISC-ASD (1989), Chapter I2, Item 1.

13. **Loading Conditions for Composite Construction Using Shear Connectors.** Composite construction using shear connectors should be designed for the various loading conditions in Section 2208, AISC-ASD (1989), Chapter I2, Item 2.

14. **Horizontal Design Shear Transferred by Shear Connectors.** For full composite action with concrete subject to flexural compression, the total horizontal shear to be resisted between the point of maximum positive moment and points of zero moment should be taken as the smaller value using Equations (I4-1) and (I4-2). [AISC-ASD (1989), Chapter I4]

15. **Number of Shear Connectors for Full Composite Action.** For full composite action, the number of connectors resisting the horizontal shear V_h each side of the point of maximum moment should not be less than that determined by the relationship V_h/q. [AISC-ASD (1989), Chapter I4]

16. **Placement of Shear Connectors.** Shear connectors should be placed in accordance with the following limitations of AISC-ASD (1989), Chapter I4:

    a.   Shear connectors should have a least 1 inch (25 mm) of lateral concrete cover, except for connectors installed in the ribs of formed steel decks.

    b.   Unless located directly over the web, the diameter of studs should not be greater than $2^1/_2$ times the thickness of the flange to which they are welded.

    c.   The minimum center-to-center spacing of stud connectors should be 6 diameters along the longitudinal axis of the supporting composite beam.

d. The maximum center-to-center spacing of stud connectors should not exceed eight times the total slab thickness.

COMPOSITE BEAMS WITH FORMED STEEL DECK

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17. **General Limitations.** The following limitations apply to the design of composite beams with formed steel deck in accordance with Section 2208, AISC-ASD (1989), Chapter I5:

   a. The nominal rib height of the deck should not exceed 3 inches (76 mm).

   b. The average width of concrete rib or haunch should not be less than 2 inches (51 mm).

   c. The concrete slab should be connected to the steel beam or girder with welded stud shear connectors $^3/_4$ inch (19 mm) or less in diameter.

   d. Stud shear connectors should extend not less than $1^1/_2$ inches (38 mm) above the top of the steel deck after installation.

   e. The slab thickness above the steel deck should not be less than 2 inches (51 mm).

18. **Provisions for Deck Ribs Oriented Perpendicular to Steel Beam or Girder.** The following provisions of AISC-ASD (1989), Chapter I5, Item 2, should be met for composite construction where the steel deck ribs are oriented perpendicular to the steel beam or girder:

   a. Concrete below the top of the steel deck should be neglected when determining section properties and in calculating A_c for Equation (I4-1).

   b. The spacing of stud shear connectors should not exceed 36 inches (914 mm).

   c. The allowable horizontal shear load per stud connector q should be multiplied by the reduction factor determined by Equation (I5-1).

   d. To resist uplift, the steel deck should be anchored to compositely-designed steel beams or girders at a spacing not to exceed 16 inches (406 mm).

19. **Provisions for Deck Ribs Oriented Parallel to Steel Beam or Girder.** The following provisions of AISC-ASD (1989), Chapter I5, Item 3, should be met for composite construction where the steel deck ribs are oriented parallel to the steel beam or girder:

   a. Concrete below the top of the steel deck should be included in calculating A_c for Equation (I4-1).

b. When the nominal depth of steel deck is $1^1/_2$ inches (38 mm) or greater, the average width of the supported haunch or rib should not be less than 2 inches (51 mm) for the first stud in the transverse row, plus four stud diameters for each additional stud.

c. The allowable horizontal shear load per stud connector q should be the value stipulated in Section I4 except when the ratio w_r/h_r is less than 1.5, in which case the allowable load should be multiplied by the reduction factor determined by Equation (15-2).

<div style="border:1px solid black; text-align:center; font-weight:bold;">WELDS</div>

20. **Minimum Effective Throat Thickness of Partial-penetration Welds.** The minimum effective throat thickness of a partial-penetration groove weld should not be less than specified in Table J2.3. [AISC-ASD (1989), Chapter J2, Item 1b]

21. **Minimum Size of Fillet Welds.** The minimum size of fillet welds should not be less than specified in Table J2.4. [AISC-ASD (1989), Chapter J2, Item 2b]

22. **Maximum Size of Fillet Welds.** The maximum size of fillet welds along edges of connected parts should be [AISC-ASD (1989), Chapter J2, Item 2b]:

a. For material less than $1/_4$ inch (6.4 mm) thick, not greater than the thickness of the material.

b. For material $1/_4$ inch (6.4 mm) or more in thickness, not greater than the thickness of the material minus $1/_{16}$ inch (1.6 mm).

23. **Minimum Effective Length of Fillet Welds.** The minimum effective length of fillet welds designed on the basis of strength should not be less than four times the nominal size, or else the size of the weld should be considered not to exceed one-fourth of its effective length. [AISC-ASD (1989), Chapter J2, Item 2b].

24. **Limitations on Plug Welds.** The following limitations for plug welds should be met [AISC-ASD (1989), Chapter J2, Item 3b]:

a. The diameter of the hole for a plug weld should not be less than the thickness of the part containing it, plus $5/_{16}$ inch (7.9 mm) rounded to the next larger odd $1/_{16}$ inch (1.6 mm), nor greater than the minimum diameter plus $1/_8$ inch (3.2 mm) or $2^1/_4$ times the thickness of the weld.

b. The minimum center-to-center spacing of plug welds should be four times the diameter of the hole. The minimum center-to-center spacing in a longitudinal direction on any line should be two times the length of the slot.

c. The thickness of plug welds in material $5/_8$ inch (16 mm) or less in thickness should be equal to the thickness of the material. In material over $5/_8$ inch (16 mm)

thick, the thickness of the weld should be at least one-half the thickness of the material but not less than $^5/_8$ inch (16 mm).

25. **Allowable Stresses in Welds.** Welds should be proportioned to meet the stress requirements set forth in Table J2.5. [AISC-ASD (1989), Chapter J2, Item 4]

<div style="border:1px solid">

BOLTS

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26. **Allowable Tension and Shear.** Allowable tension and shear stresses on bolts should not exceed those set forth in Table J3.2, in ksi (N/mm^2), of the un-threaded nominal body area of bolts. [ASIC-ASD (1989), Chapter J3, Item 4)]

27. **Combined Tension and Shear in Bearing-type Connections.** Bearing-type bolts, in combined tension and shear, should be so proportioned that the tension stress F_t in ksi (N/mm^2) on the nominal body area A_b produced by forces applied to the connected parts should not exceed the values computed from the equations in Table J3.3, where f_v, the shear stress produced by the same forces, should not exceed the value for shear given in Table J3.2. [AISC-ASD (1989), Chapter J3, Item 5)]

28. **Combined Tension and Shear in Slip-critical Joints.** For A 325 and A 490 bolts used in slip-critical connections, the maximum shear stress allowed by Table J3.2 should be multiplied by the reduction factor $(1 - f_t A_b / T_b)$. [AISC-ASD (1989), Chapter J3, Item 6]

29. **Minimum Spacing.** The distance between centers of standard, oversized or slotted fastener holes should not be less than $2^2/_3$ times the nominal diameter of the fastener or less than that required by AISC-ASD (1989), Chapter J3, Item 8.

30. **Minimum Edge Distance.** The distance from the center of a standard hole to an edge of a connected part should not be less than the appropriate value from Table J3.5 or from Equation (J3-6), as applicable. [AISC-ASD (1989), Chapter J3, Item 9]

31. **Maximum Edge Distance and Spacing.** The maximum distance from the center of any rivet or bolt to the nearest edge of parts in contact should be 12 times the thickness of the connected part under consideration, but should not exceed 6 inches (152 mm). [AISC-ASD (1989), Chapter J3, Item 10]

32. **Eccentric Connections.** Where intersecting axially-stressed members do not have their gravity axes intersecting at one point, provision should be made for bending and shearing stresses due to their eccentricity. [AISC-ASD (1989), Chapter J5, Item 1]

I. Seismic Design Requirements (Seismic Zones 3 and 4)

1. **Materials (Seismic Zones 3 and 4).** Structural steel used in lateral-force-resisting systems should conform to A 36, A 500, A 501, A 572 (Grades 42 and 50), A913 (Grades 50 and 65) or A 588, except as noted in Section 2213.4.1.

IX STEEL

2. **Column Strength (Seismic Zones 3 and 4).** Columns in frames should have the strength (see Section 2213.4.2 for definition of strength) to resist the axial loads resulting from the following load combinations in accordance with Section 2213.5.1:

   Axial Compression: $1.0\ P_{DL} + 0.7\ P_{LL} + \Omega_o P_E$
   Axial Tension: $0.85\ P_{DL} \pm \Omega_o P_E$

3. **Column Splices (Seismic Zones 3 and 4).** Column splices should have sufficient strength to develop the column forces determined from the following load combinations in accordance with Section 2213.5.2:

   Axial Compression: $1.0\ P_{DL} + 0.7\ P_{LL} + \Omega_o P_E$
   Axial Tension: $0.85\ P_{DL} \pm \Omega_o P_E$

4. **Welded Column Splices (Seismic Zones 3 and 4).** Welded column splices subject to net tensile forces should comply with the more critical of the following (Section 2213.5.2):

   a. Partial penetration welds should be designed to resist 150 percent of the axial tension force determined by the $0.85\ P_{DL} \pm \Omega_o P_E$ load combination.

   b. Welding should develop not less than 50 percent of the flange area strength of the smaller column.

5. **Column Splices With Partial Penetration Welds (Seismic Zones 3 and 4).** Splices employing partial penetration welds should be located at least 3 feet (914 mm) from girder flanges. (Section 2213.5.2)

6. **OMRF Connection Requirement (Seismic Zones 3 and 4).** Girder-to-column connections of OMRF's should meet the requirements of Section 2213.7 unless it can be shown that they are capable of resisting the combination of gravity loads and Ω_o times the design seismic forces. Alternatively, partially restrained connections may be used provided the requirements of Section 2213.6, Item 3, are met. (Section 2213.6)

IX STEEL

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7. **Girder-to-column Connection Strength (Seismic Zones 3 and 4).** Girder-to-column connections in SMRFs should be shown, by approved cyclic test results or calculation, to have the ability to sustain inelastic rotation and develop the strength criteria of Section 2213.7.1.1, considering the effect of steel overstrength and strain hardening. See Section 2213.4.2 for member strengths. (Section 2213.7.1.2)

8. **Steel with F_y/F_u < 1.5 (Seismic Zones 3 and 4).** For steel whose specified ultimate strength is less than 1.5 times the specified yield strength, plastic hinges should not form at locations in which the beam flange area has been reduced, such as for bolt holes. (Section 2213.7.1.3)

9. **Bolted Flange Plates of Beam Column Joints (Seismic Zones 3 and 4).** SMRF bolted connections of flange plates of beam-column joints should have the net-to-gross area ratio A_e/A_g equal to or greater than 1.2 F_y/F_u. (Section 2213.7.1.3)

10. **Required Panel Zone Strength (Seismic Zones 3 and 4).** The panel zones of the SMRF joints should have the strength to resist the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. See Section 2213.4.2 for member strengths. (Section 2213.7.2.1)

11. **Minimum Panel Zone Thickness (Seismic Zones 3 and 4).** The panel zone thickness t_z should conform to the following formula:

$$t_z \geq (d_z + w_z)/90 \qquad\qquad \text{Formula (13-2)}$$

    For this purpose, t_z should not include any double plate thickness unless the doubler plate is connected to the column web with plug welds. (Section 2213.7.2.2)

12. **Doubler Plates (Seismic Zones 3 and 4).** Doubler plates provided to reduce SMRF panel zone shear stress or to reduce the web depth thickness ratio should be placed not more than $^1/_{16}$ inch (1.6 mm) from the column web and should be welded across the plate width, top and bottom, with at least a $^3/_{16}$-inch (4.7 mm) fillet weld. They, should be either butt or fillet welded to the column flanges to develop the shear strength of the doubler plate. Weld strength should be as given in Section 2213.4.2. (Section 2213.7.2.3)

13. **Maximum Flange Width-thickness Ratio (Seismic Zones 3 and 4).** The flange width-thickness ratio of SMRF beams $b_f / 2t_f$ should not exceed $52/\sqrt{F_y}$ (For **SI**: $0.31/\sqrt{E/F_y}$). The width-thickness ratio of SMRF column sections should meet the requirements of Division III, Section 2251, N7 (i.e., $b_f/2t_f$ should not exceed 8.5 for

F_y = 36 ksi and 7.0 for F_y = 50 ksi). The outside wall width-thickness ratio of rectangular tubes used for SMRF columns should not exceed $110/\sqrt{F_y}$ (For **SI**: $0.65/\sqrt{E/F_y}$) unless otherwise stiffened. (Section 2213.7.3)

14. **Continuity Plates (Seismic Zones 3 and 4).** When determining the need for SMRF girder tension flange continuity plates, the value of P_{bf} in Division III should be taken as $1.8(bt_f)F_{yb}$. (Section 2213.7.4)

15. **Strong Column-Weak Beam Strength Ratio (Seismic Zones 3 and 4).** At any SMRF frame joint, the following relationships should be satisfied except as noted in Section 2211.7.5:

$$\sum Z_c(F_{yc} - f_a)/\sum M_c > 1.0 \qquad \text{Formula (13-3-1)}$$

$$\sum Z_c(F_{yc} - f_a)/1.25\sum M_{pz} > 1.0 \qquad \text{Formula (13-3-2)}$$

16. **Girder-column Joint Restraint for Columns Expected to Remain Elastic (Seismic Zones 3 and 4).** Where it can be shown that the columns of SMRFs remain elastic, the flanges of the columns need be laterally supported only at the level of the girder top flange. The column flange lateral support should be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses and at a limiting displacement perpendicular to the frame of 0.2 inch (5.1 mm). (Section 2213.7.7.1)

17. **Girder-column Joint Restraint for Columns Not Expected to Remain Elastic (Seismic Zones 3 and 4).** Where the columns of SMRFs cannot be shown to remain elastic, the column flanges should be laterally supported at the levels of the girder top and bottom flanges. The column flange lateral support should be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses and at a limiting displacement perpendicular to the frame of 0.2 inch (5.1 mm). (Section 2213.7.7.1)

18. **SMRF Beam Bracing (Seismic Zones 3 and 4).** Both flanges of SMRF beams should be braced directly or indirectly. The beam bracing between column center lines should not exceed $96r_y$. In addition, braces should be placed at concentrated loads where a hinge may form. (Section 2213.7.8)

19. **Changes in SMRF Beam Flange Area (Seismic Zones 3 and 4).** Abrupt changes in beam flange area should not occur within possible plastic hinge regions of SMRFs. (Section 2213.7.9)

20. **SMRF Drift Calculations (Seismic Zones 3 and 4).** SMRF drift calculations should include bending and shear contributions from the clear girder and column spans, column axial deformation and the rotation and distortion of the panel zone. (Section 2213.7.10)

IX STEEL

21. **Nondestructive Testing Requirements (Seismic Zones 3 and 4).** A nondestructive testing program in accordance with Section 1703 should be defined on the plans for testing welded SMRF connections.

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CONCENTRICALLY BRACED FRAMES
(SEISMIC ZONES 3 AND 4)

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22. **Maximum Slenderness Ratio for Bracing Members (Seismic Zones 3 and 4).**
The l/r for bracing members should not exceed $720/\sqrt{F_y}$ (For **SI**: $4.23\sqrt{E/F_y}$). (Section 2213.8.2.1)

23. **Stress Reduction Factor for Bracing Members in Compression (Seismic Zones 3 and 4).** The allowable stress F_{as} for bracing members resisting seismic forces in compression should be determined from the following formula (Section 2213.8.2.2):

$$F_{as} = BF_a \qquad\qquad \text{Formula (13-4)}$$

$$B = 1/\{1 + [(Kl/r)/2C_c]\} \qquad\qquad \text{Formula (13-5)}$$

24. **Lateral Force Distribution in Braced Frames (Seismic Zones 3 and 4).** The seismic lateral force along any line of bracing should be distributed to the various members so that neither the sum of the horizontal components of the forces in members acting in tension nor the sum of the horizontal components of forces in members acting in compression exceed 70 percent of the total force, except as noted in Section 2213.8.2.3.

25. **Design Forces for Bracing Connections (Seismic Zones 3 and 4).** Bracing connections should have the strength (see Section 2213.4.2 for member strength definition) to resist the lesser of the following: (Section 2213.8.3.1)

   a.   The strength of the bracing in axial tension, P_{st}.

   b. Ω_o times the force in the brace due to the prescribed seismic forces, in combination with gravity loads.

   c.   The maximum force that can be transferred to the brace by the system.

26. **Eccentricities in Bracing Connections (Seismic Zones 3 and 4).** Eccentricities in the frame geometry or connection load path should be considered in accordance with Section 2213.8.3.1.

27. **Minimum Net Area in Bolted Brace Connections (Seismic Zones 3 and 4).** In bolted brace connections, the ratio of effective net section area to gross section area should satisfy the following (Section 2213.8.3.2):

$$\frac{A_e}{A_g} \geq \frac{1.2\alpha F^*}{F_u} \qquad\qquad \text{Formula (13-6)}$$

28. **Chevron Bracing Requirements (Seismic Zones 3 and 4).** Chevron bracing should be shown to comply with the following requirements (Section 2213.8.4.1):

    a. Bracing members should be designed for 1.5 times the otherwise prescribed seismic forces.

    b. The beam intersected by chevron braces should be continuous between columns.

    c. Where chevron braces intersect a beam from below, the beam should be capable of supporting all tributary gravity loads presuming the bracing not to exist.

29. **K Bracing Limitations (Seismic Zones 3 and 4).** K bracing is prohibited except in buildings not over two stories in height and in roof structures, if the braces have the strength to resist Ω_o times the design seismic forces. (Section 2213.8.4.2)

> ## SPECIAL CONCENTRICALLY BRACED
> ## FRAMES (SEISMIC ZONES 2, 3 AND 4)

NOTE: The requirements of Section 2213.8 for concentrically braced frames generally apply as listed in the previous section except as noted below.

30. **Maximum Slenderness Ratio for Bracing Members (Seismic Zones 2, 3 and 4).** The kl/r for bracing members should not exceed $1,000/\sqrt{F_y}$ (For **SI**: $5.87\sqrt{E/F_y}$). (Section 2213.9.2.1)

31. **Maximum Width-thickness Ratio of Compression Elements (Seismic Zones 2, 3 and 4).** The width-thickness ratio of compression elements used in braces should not exceed the limitations set forth in Section 2213.9.2.4.

32. **Strength of Brace End Connections (Seismic Zones 2, 3 and 4).** End connections of braces should provide a flexural strength in excess of that of the brace gross section about the critical buckling axis. (Section 2213.9.3.3)

33. **Additional Chevron Bracing Requirements (Seismic Zones 2, 3 and 4).** The beam intersected by chevron braces should have the strength to support the load combinations set forth in Section 2213.9.4.1, Item 3. Also, both flanges of beams at the point of intersection of chevron braces should be laterally supported in accordance with Section 2213.9.4.1, Item 4.

34. **No K Bracing (Seismic Zones 2, 3 and 4).** K bracing should not be used. (Section 2213.9.4.2)

35. **Column Splices (Seismic Zones 2, 3, and 4).** Column splices should be designed to develop the full shear strength and 50 percent of the full moment strength of the

section. Also, column splices should be located in the middle one-third of the column clear height. (Section 2213.9.5)

ECCENTRICALLY BRACED FRAMES (EBF)
(SEISMIC ZONES 2, 3 AND 4)

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36. **Maximum Flange Width-thickness Ratio for EBF Beams (Seismic Zones 2, 3 and 4).** The flange width-thickness ratio of SMRF beams, $b_f/2t_f$, should not exceed

$$52/\sqrt{F_y} \text{ (For \textbf{SI}: } 0.31\sqrt{E/F_y} \text{). (Section 2213.10.2)}$$

37. **Link Beam Requirements (Seismic Zones 2, 3 and 4).** The following requirements for link beams should be met:

    a. **Link Beam Rotation.** The rotation of the link segment relative to the rest of the beam at a total frame drift of Δ_m should not exceed the limitations set forth in Section 2213.10.4.

    b. **Link Beam Web.** The web of the link beam should be single thickness without doubler plate reinforcement. No openings should be placed in the web of a link beam. The web shear should not exceed $0.8V_s$ under prescribed lateral forces. (Section 2213.10.5)

    c. **Link Beam Stiffeners.** Link beams should have full-depth web stiffeners as set forth in Sections 2213.10.7 through 2213.10.11.

38. **Brace-to-beam Connections (Seismic Zones 2, 3 and 4).** The EBF brace-to-beam connections should develop the compression strength of the brace and transfer this force to the beam web. No part of the brace-to-beam connection should extend into the web area of a link beam. (Section 2213.10.6)

39. **Link Beam Length (Seismic Zones 2, 3 and 4).** The length of link beam connected to columns should not exceed $1.6\ M_s/V_s$. (Section 2213.10.12)

40. **Link Beam to Column Flange Connection (Seismic Zones 2, 3 and 4).** Where a link beam is connected to the column flange, the beam flange should have full-penetration welds to the column. (Section 2213.10.12, Item 1)

41. **Link Beam to Column Web Connection (Seismic Zones 2, 3 and 4).** Where the link beam is connected to the column web, the beam flanges should have full-penetration welds to the connection plates and the web connection should be welded to develop the link beam web shear strength. Also, rotation between the link beam and the column should not exceed 0.015 radians at a total frame drift of Δ_m. (Section 2213.10.12, Item 2)

42. **Brace and Beam Strengths (Seismic Zones 2, 3 and 4).** Each EBF brace and beam outside the link should have axial strength or reduced flexural strength, M_{rs}, at least

1.5 times the axial force corresponding to the controlling link beam strength (see Section 2213.4.2 for member strength definition). Columns should be designed to remain elastic at 1.25 times the strength of the EBF bay. (Sections 2213.10.13 and 2213.10.14)

43. **Bracing of Beam Flanges (Seismic Zones 2, 3 and 4).** Top and bottom flanges of EBF beams should be laterally braced at the ends of link beams and at intervals not exceeding $76/\sqrt{F_y}$ (For **SI**: $0.45\sqrt{E/F_y}$) times the beam flange width. End bracing should be designed to resist 6.0 percent of the beam flange strength, defined as $F_y\, b_f\, t_f$. Intermediate bracing should be designed to resist 1.0 percent of the beam flange force at the brace point, using the link beam strength determined in Section 2213.10.13. (Section 2213.10.18)

> **STUD WALL SYSTEMS**
> **(SEISMIC ZONES 3 AND 4)**

44. **Limitations on Stud Wall Systems (Seismic Zones 3 and 4).** Stud wall systems should not be used to resist the specified seismic forces in buildings over five stories in height. (Section 2220.1)

45. **Requirements for Stud Wall Systems (Seismic Zones 3 and 4).** Stud wall systems designed to resist seismic forces should meet the following requirements of Section 2220.1 in addition to the requirements of Section 2219.

    a. Boundary members, chords and collectors should be designed and detailed to transmit the induced axial forces.

    b. Connection of the diagonal bracing member, top chord splices, boundary members and collectors should be designed to develop the full tensile strength of the member or Ω_o times the otherwise prescribed seismic forces.

    c. Vertical and diagonal members of the braced bay should be anchored so the bottom track is not required to resist uplift forces by bending of the track web.

    d. Both flanges of studs in a bracing panel should be braced to prevent lateral torsional buckling.

    e. Screws should not be used to resist lateral forces by pullout resistance.

    f. Provision should be made for pretensioning or other methods of installation of tension-only bracing to guard against loose diagonal straps.

IX STEEL

46. ***W* Used in Design of Storage Racks (Seismic Zones 3 and 4).** The *W* used in the design of racks over 8 feet (2438 mm) in height using the provisions of Chapter 22, Division X, in Seismic Zones 3 and 4 in wholesale and retail sales areas should be equal to the weight of the rack structure and contents with no reductions. (Section 2205.8)

47. **Seismic Design for Storage Racks.** Storage racks over 6 feet in height may be designed for seismic forces in accordance with Chapter 22, Division X, provided the seismic design forces are equal to or greater than those specified in Section 1632.2 or 1634.2, as appropriate. (Table 16-O, Footnote 4)

MATERIAL REQUIREMENTS IN SEISMIC ZONE 2

48. **Materials (Seismic Zone 2).** Structural steel used in lateral-force-resisting systems should conform to A 36, A 500, A 501, A 572 (Grades 42 and 50) or A 588, except as noted in Sections 2213.4 and 2214.3.

ORDINARY MOMENT-RESISTING FRAME
REQUIREMENT (SEISMIC ZONE 2)

49. **OMRF Connection Requirement (Seismic Zone 2).** Girder-to-column connections of OMRFs should meet the requirements of Section 2214.5.1 unless it can be shown that they are capable of resisting the combination of gravity loads and Ω_o times the design seismic forces. Alternatively, partially restrained connections may be used provided the requirements of Section 2214.4, Item 3, are met. (Section 2214.4)

SPECIAL MOMENT-RESISTING FRAME
REQUIREMENTS (SEISMIC ZONE 2)

50. **Girder-to-column Connection Strength (Seismic Zone 2).** The SMRF girder-to-column connection should be shown to have the required strength in accordance with Section 2214.5.1.

51. **Steel with $F_y/F_u < 1.5$ (Seismic Zone 2).** For steel whose specified ultimate strength is less than 1.5 times the specified yield strength, plastic hinges should not form at locations in which the beam flange area has been reduced, such as for bolt holes. (Section 2214.5.1.4)

52. **Bolted Flange Plates of Beam-Column Joints (Seismic Zone 2).** SMRF bolted connections of flange plates of beam-column joints should have a net-to-gross area ratio A_e/A_g equal to or greater than 1.25 F_y/F_u. (Section 2214.5.1.4)

53. **Girder-column Joint Restraint for Columns Expected to Remain Elastic (Seismic Zone 2).** Where it can be shown that the columns of SMRFs remain elastic, the flanges of the columns need be laterally supported only at the level of the girder top flange. The column flange lateral support should be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses and at a limiting displacement perpendicular to the frame of 0.2 inch (5.1 mm). (Section 2214.5.3.1)

54. **Girder-column Joint Restraint for Columns Not Expected to Remain Elastic (Seismic Zone 2).** Where the columns of SMRFs cannot be shown to remain elastic, the column flanges should be laterally supported at the levels of the girder top and bottom flanges. The column flange lateral support should be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses and at a limiting displacement perpendicular to the frame of 0.2 inch (5.1 mm). (Section 2214.5.3.1)

55. **Changes in SMRF Beam Flange Area (Seismic Zone 2).** Abrupt changes in beam flange area should not occur within possible plastic hinge regions of SMRFs. (Section 2214.5.4)

CONCENTRICALLY BRACED FRAMES
(SEISMIC ZONE 2)

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56. **Stress Reduction Factor for Bracing Members in Compression (Seismic Zone 2).** The allowable stress F_{as} for bracing members resisting seismic forces in compression should be determined from the following formula: (Section 2214.6.2.1)

$$F_{as} = BF_a \qquad \text{Formula (14-1)}$$
$$B = 1/\{1 + [(Kl/r)/2C_c]\} \geq 0.8 \qquad \text{Formula (14-2)}$$

57. **Design Forces for Bracing Connections (Seismic Zone 2).** Bracing connections should be designed for the lesser of the following: (Section 2214.6.3.1)

   a. The tensile strength of the bracing.

   b. Ω_o times the force in the brace due to prescribed seismic forces.

   c. The maximum force that can be transferred to the brace by the system.

58. **Minimum Net Area in Bolted Brace Connections (Seismic Zone 2).** In bolted brace connections, the ratio of effective net section area to gross section area should satisfy the following: (Section 2214.6.3.2)

$$\frac{A_e}{A_g} \geq \frac{1.2\alpha F^*}{F_u}$$ Formula (14-3)

59. **Chevron and K Bracing Requirements (Seismic Zone 2).** Chevron and K bracing should be shown to comply with the following requirements: (Section 2214.6.4)

   a. Bracing members should be designed for 1.5 times the otherwise prescribed forces.

   b. The beam intersected by chevron braces should be continuous between columns.

   c. Where chevron braces intersect a beam from below, the beam should be capable of supporting all tributary gravity loads presuming the bracing not to exist.

60. **Nondestructive Testing Requirements.** A nondestructive testing program in accordance with Section 1703 should be defined on the plans for testing welded SMRF connections. (Section 2214.9)

X   WOOD

X. WOOD

A.   Material Requirements

1.   **Grading Rules.** Plans should specify grading rules by which lumber is to be graded. (Section 2304)

2.   **Specifications for Glulams.** Plans should specify the following for structural glued-laminated timber: (Section 2316.1 and NDS-91, Section 5.1)

    a.   Whether for dry or wet conditions of use.

    b.   Species and applicable standard.

    c.   Stress requirements.

    d.   If the temperature of the timber exceeds 150°F (66°C) in service.

3.   **Exposed Structural Glulams.** Exposed structural glulams should be pressure treated with an approved preservative or be manufactured from wood of natural resistance to decay. (Section 2306.12)

4.   **Specifications for Wood Structural Panels.**

    a.   **Grade Specification.** Plans should specify wood structural panel grade. Wood structural panel grade should be one of the grades specified in UBC Standard 23-2 or 23-3. (Sections 2304.1 and 2315.3.3)

    b.   **Wood Structural Panels Used Outdoors**. Wood structural panels to be exposed in outdoor applications should be of the exterior type, except as provided in Sections 2312.2 and 2320.12.9. (Section 2310.3)

    c.   **Wood Structural Panel Shear Walls and Diaphragms.** Plywood to be used in shear-walls and diaphragms should be manufactured using exterior glue. (Section 2315.5.3)

5.   **Wood Subject to Decay and Termites**. Protection against decay and termites should be specified for wood in accordance with Section 2306.

B.   Adjustment Factors

1.   **Applicable Adjustment Factors.** Tabulated allowable design values should be multiplied by all applicable adjustment factors as set forth in Sections 2316 and 2316.2, and NDS-91, Part II. (Division III - Part I)

2.   **Fire-retardant Treated Wood**. Allowable stresses for fire-retardant treated wood should be reduced in accordance with Sections 2316, 2316.2 and UBC Standard 23-5.

X   WOOD

3. **Fire-retardant Treated Wood Exposed to Higher Temperatures.** The allowable unit stresses for fire-retardant-treated solid-sawn lumber and plywood, including fastener values, subject to prolonged elevated temperatures from manufacturing or equipment processes not exceeding 150°F (66°C) should be determined in accordance with Sections 2316 and 2316.2.

4. **Duration of Load.** The duration of load adjustments set forth in NDS-91 shall be in accordance with Sections 2316 and 2316.2.

5. **Duration of Load.** The duration of load factors for compression-perpendicular-to-grain design values in sawn lumber shall be based on NDS-91, Part IV. (Section 2316)

6. **Load Duration Factor C_D.** Duration of load increases for wind, earthquake, roof live load, snow and impact shall be in accordance with Table 2.3.2. (Section 2316.2)

7. **Size Factor Adjustment for 2x, 3x and 4x Members.** Bending, tension and compression parallel to grain design values for visually graded 2x, 3x and 4x members should be multiplied by the size factors specified in Table 4A, NDS-91, Section 4.3.2, as indicated in Table X-1. (Section 2316.1)

TABLE X-1
SIZE FACTORS, C_F

GRADES	WIDTH (inches) x 25.4 for mm	F_b Thickness 2" and 3" (51 mm and 76 mm)	F_b Thickness 4" (102 mm)	F_t	F_c
Select structural, No. 1 and better No. 1, No. 2, No. 3	2, 3 and 4	1.5	1.5	1.5	1.15
	5	1.4	1.4	1.4	1.1
	6	1.3	1.3	1.3	1.1
	8	1.2	1.3	1.2	1.05
	10	1.1	1.2	1.1	1.0
	12	1.0	1.1	1.0	1.0
	14 and wider	0.9	1.0	0.9	0.9
Stud	2, 3 and 4	1.1	1.1	1.1	1.05
	5 and 6	1.0	1.0	1.0	1.0
Construction and Standard	2, 3 and 4	1.0	1.0	1.0	1.0
Utility	4	1.0	1.0	1.0	1.0
	2 and 3	0.4	—	0.4	0.6

8. **Size Factor Adjustment for 5 Inches (127 mm) and Thicker Members with a Depth Exceeding 12 Inches (305 mm).** When the depth of a rectangular sawn lumber bending member 5 inches (127 mm) or thicker exceeds 12 inches (305 mm), the bending design value F_b should be multiplied by the size factor C_F, as determined by the formula (NDS-91, Section 4.3.2):

$$C_F = (12/d)^{1/9} \leq 1.0$$

For **SI**: $C_F = (304.8/d)^{1/9}$

9. **Size Factor and Slenderness Factor.** The volume factor C_V adjustment should not be cumulative with the beam stability factor C_L for glued-laminated timber bending members. (NDS-91, Section 5.3.2)

10. **Temperature.** Allowable design stresses should be decreased in accordingly with wood subject to prolonged elevated temperatures from manufacturing or equipment processes but not exceeding 150°F (66°C). (Sections 2316 and 2316.2).

11. **Moisture Service Conditions — Sawn Lumber and Fastenings.** Allowable design values for sawn lumber and fastenings exposed to service conditions causing the wood to possess more than 19 percent moisture content should be reduced as specified in NDS-91, Section 4.1.4. (Section 2316).

12. **Moisture Service Conditions — Glulams.** Where glued-laminated timber exceeds 16 percent moisture content, the tabulated design values should be reduced as specified in NDS-91, Section 5.1.5. (Section 2316).

C. Beam Design

1. **Span Length, Simple Beams.** The span should be the distance from face-to-face of supports plus one-half the required length of bearing at each end. (NDS-91, Section 3.2)

2. **Span Length, Continuous Beams.** The span should be the distance between centers of bearings on supports over which the beam is continuous. (Section 2316.2; NDS-91, Section 3.2)

3. **Notching.** Notching of beams should be avoided wherever possible. If notching is absolutely necessary, the following limitations should be adhered to (Section 2316.2; NDS-91, Section 3.2.3):

   a. Notches in sawn lumber should not exceed one-sixth the depth of the member and should not be located in the middle third of the span.

   b. Where members are notched at the ends, the notch depth should not exceed one-fourth the beam depth.

   c. The tension side of sawn lumber bending members of 4 inches (102 mm) or greater nominal thickness should not be notched except at ends of members for bearing over a support.

   d. Cantilevered portions of beams less than 4 inches (102 mm) in nominal thickness should not be notched unless the reduced section properties and lumber defects are considered in the design.

e.   Girders, beams or joists notched at points of support should be shown to meet design requirements for that net section in bending and in shear. The shear at notching should be shown to be within the values set forth in NDS-91, Section 3.4.4.

4.   **Cross-grain Bending.** Wood should not be used in cross-grain bending, as there are no allowable design values. Where designs that induce tension stresses perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist such forces should be provided. (Section 2316.2; NDS-91, Section 3.8.2)

5.   **Lateral Support.** Solid-sawn rectangular lumber beams, rafters and joists should be supported laterally in accordance with the prescriptive requirements set forth in NDS-91, Section 4.4.1.2, or the allowable stresses should be reduced by the slenderness factor set forth in NDS-91, Section 3.3.3. (Section 2316.2; NDS-91, Section 4.4.1)

D.   Column Design

1.   **Spaced columns.** Spaced columns should be designed in accordance with Section 2316; NDS-91, Part XV.

2.   **Limitation on l/d Ratios for Simple Solid Columns.** For simple solid columns, l_e/d should not exceed 50. (Section 2316.2; NDS-91, Section 3.7)

3.   **Effective Column length.** A design buckling factor of _____ should be used to determine the effective column length in accordance with NDS-91, Section 3.7 and Appendix G.

4.   **Combined Stresses.** Combined flexure and axial compression should be checked in accordance with NDS-91, Section 3.9.2.

E.   Bearing at Angle to Grain. The allowable bearing design value at an angle of load to grain between 0 and 90 degrees should be computed from Hankinson's Formula in accordance with NDS-91, Section 3.10.3.

F.   Timber Connectors and Fasteners

1.   **Fasteners for Pressure-treated and Fire-retardant-treated Wood.** Fasteners for pressure-preservative-treated and fire-retardant-treated wood should be of hot-dipped zinc-coated galvanized, stainless steel, silicon bronze or copper. (Section 2304.3)

2.   **Bolts**

a.   **Maximum Bolt Diameter in Wood Connections.** Bolt diameters in wood connections should be no more than 1 inch (25 mm) in diameter. (Section 2316.2 and NDS-91, Section 8.1.1)

b. **Allowable Shear.** Allowable shear loads for bolts in seasoned lumber of Douglas-fir-larch and southern pine should not exceed the values set forth in Tables 23-III-B-1 and 23-III-B-2. (Section 2318.2)

c. **Allowable Shear for Wood Members Connected to Concrete or Masonry.** Allowable shear for bolts connecting wood members to concrete or masonry should be determined as one-half the tabulated double shear values for a wood member twice the thickness of the member attached to the concrete or masonry. (Section 2316.2; NDS-91, Section 8.3)

d. **End Distances.** End distances for bolt holes should be provided in accordance with NDS-91, Section 8.5.4.

e. **Spacing.** Bolt spacing should be provided in accordance with NDS-91, Section 8.5.5.

f. **Edge Distances.** Edge distances for bolt holes should be provided in accordance with NDS-91, Section 8.5.3.

3. **Nails**

a. **Type of Nail.** Plans should specify common nails (NOT box nails) are to be used, as the allowable values assumed in the calculations for nails is for common nails as set forth in Table 23-III-C-2. (Section 2318.3.1)

b. **Nails Parallel to Grain.** Nails driven parallel to the grain of the wood should not be subjected to more than two-thirds of the lateral load allowed per Tables 23-III-C-1 and 23-III-C-2. (Section 2318.3.1)

c. **Toenails.** Toenails should not be subjected to more than five-sixths of the lateral load allowed per Tables 23-III-C-1 and 23-III-C-2. (Section 2318.3.1)

d. **Withdrawal Value.** Allowable withdrawal values for common wire and box nails should not exceed those set forth in Table 23-III-D. (Section 2318.3.2)

e. **No Withdrawal Value for Parallel-to-grain.** Nails driven parallel to grain should not be allowed to resist withdrawal forces. (Section 2318.3.2)

f. **Spacing of Nails.** For wood-to-wood joints, the spacing center-to-center of nails in the direction of stress should not be less than the required penetration (Section 2316.2; NDS-91, Section 12.4.1.)

g. **Edge or End Distance.** Edge or end distances in the direction of stress should not be less than one-half of the required penetration (Section 2316.2; NDS-91, Section 12.4.1.)

4. **Lag Screws**

a. **Allowable Loads.** Allowable loads for lag screws should be determined in accordance with Section 2316 and NDS-91, Part IX.

b.   **Placement.** The spacings, end distances and edge distances for lag screws should be the same as for bolts of a diameter equal to the shank diameter of the lag screw used. (NDS-91, Section 9.4)

G.   Glulam Design

1.   **Curvature Factor.** For the curved portion of members, the allowable bending stress should be reduced in accordance with NDS-91, Section 5.3.4.

2.   **Radial Stress.** The maximum radial stress in curved members should be shown to be less than the allowable values set forth in Section 2316.2 and NDS-91, Section 5.4.1.

3.   **Volume Factor.** The allowable bending stress should be multiplied by the volume factor set forth in NDS-91, Section 5.3.2.

4.   **Ponding.** In no case should the roof slope provide a positive vertical displacement less than that equivalent to $^1/_4$ unit vertical in 12 units horizontal (2% slope) of horizontal distance between the level of the drain and the high point of the roof for drainage. Such slope should be in addition to the camber in glulams of $1^1/_2$ times the calculated dead load deflection. (Sections 1506 and 2316.2; NDS-91, Section 5.4.4.)

H.   Truss Design. Metal-plate-connected wood trusses should meet the requirements of Chapter 23, Division V. (Section 2304.4.4)

I.   Wood Combined with Masonry or Concrete

1.   **No Support of Masonry and Concrete Dead Loads.** Wood members should not be used to permanently support the dead load of masonry or concrete except as noted in Section 2307.

2.   **Horizontal Forces Contributed by Masonry or Concrete.** Wood members should not be used to resist horizontal forces contributed by masonry or concrete in buildings over one story in height except as noted in Section 2315.2.

J.   Detailing for Horizontal Forces

1.   **Connections and Anchorages.** Shear transfer connections and anchorages capable of resisting the design forces should be provided between the diaphragms and the resisting elements. (Section 2315.1)

2.   **Detailing of Openings in Diaphragms.** Openings in diaphragms should be fully detailed on the plans and should have their edges adequately reinforced to transfer shearing stresses. (Section 2315.1)

3. **Diaphragm Dimension Limitations.** The size and shape of diaphragms should be limited as set forth in Table 23-II-G. (Section 2315.1)

4. **Diaphragms with Nailing $\leq 2^1/_2$ Inches (64 mm).** Plans should specify that framing at adjoining horizontal diaphragm panel edges will be 3-inch (76 mm) nominal or wider and nails will be staggered where nails are spaced 2 inches (51 mm) or $2^1/_2$ inches (64 mm) on center. (Table 23-II-H, Footnote 2)

5. **Diaphragms with 10d Nailing ≤ 3 Inches (76 mm).** Plans should specify that framing at adjoining horizontal diaphragm panel edges will be 3-inch (76 mm) nominal or wider and nails will be staggered where 10d nails having penetration into framing of more than $1^5/_8$ inches (41 mm) are spaced 3 inches (76 mm) or less on center. (Table 23-II-H, Footnote 3)

6. **Shear Walls with Panels on Both Sides.** Where panels are applied on both faces of a shear wall and nail spacing is less than 6 inches (152 mm) on center on either side, plans should either specify that panel joints will be offset to fall on different framing members or that framing will be 3-inch (76 mm) nominal or thicker and nails on each side will be staggered. (Table 23-II-I-1, Footnote 2 and Table 23-II-I-2, Footnote 2)

7. **Shear walls with 10d Nailing Spaced ≤ 3 Inches (76 mm).** Plans should specify that framing at adjoining panel edges will be 3-inch (76 mm) nominal or wider and nails will be staggered where 10d nails having penetration into framing of more than $1^5/_8$ inches (41 mm) are spaced 3 inches (76 mm) or less on center. (Table 23-II-I-2, Footnote 5)

8. **Collector Members (Seismic Zones 3 and 4).** Collector members should be provided to transmit tension and compression forces, and diaphragm sheathing should not be used to splice these members. (Section 2315.5.2)

9. **Blocked Walls (Seismic Zones 3 and 4).** Plans should specify that framing members or blocking should be provided at the edges of all sheets in shear walls. (Sections 2315.5)

10 **Toenails (Seismic Zones 3 and 4).** Except for structures built in accordance with Section 2320, toenails shall not be used to transfer lateral forces in excess of 150 pounds per foot (2188 N/m) from diaphragms to shear walls, drag struts (collectors) or other elements, or from shear walls to other elements. (Section 2318.3.1)

11. **3-Inch (76 mm) Framing for Shear Walls (Seismic Zone 3 and 4).** Where allowable shear values exceed 350 pounds per foot (5.11 N/mm), foundation still plates and all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch (76 mm) nominal member and foundation sill plates shall not be less than a single 3-inch (76 mm) nominal member. In shear walls where total wall design shear does not exceed 600 pounds per foot (8.76 N/mm), a single 2-inch (51 mm) nominal sill plate may be used, provided anchor bolts are designed for a load capacity of 50 percent or less of the allowable capacity and bolts have a minimum of 2-inch by

2-inch by $^3/_{16}$-inch-thick (51 mm by 51 mm by 5 mm) plate washers. Plywood joint and sill plate nailing shall be staggered in all cases. (Table 23-II-I-1, Footnote 3 and Table 23-II-I-2, Footnote 3)

12. **Shear Walls With Openings Designed for Force Transfer Around the Openings.** Where shear walls with openings are designed for force transfer around the openings, the limitations of Table 23-II-G shall apply to the overall shear wall including openings and to each wall pier at the side of an opening. The height of a wall pier shall be defined as the clear height of the pier at the side of an opening. The width of a wall pier shall be defined as the sheathed width of the pier at the side of an opening. Design for force transfer shall be based on a rational analysis. Detailing of boundary members around the openings shall be provided in accordance with Section 2315. [Section 2315.1 and Figure 23-II-1, Section (b)]

13. **Roof Sheathing Nailing.** Nailing for wood structured panel roof sheathing shall comply with the minimum nailing schedule as shown in Table 23-II-B-2 for (greater than 90 mph), (greater than 80 mph to 90 mph), (80 mph or less) wind region. (Section 2304.3 and Table 23-II-B-2)