CHAPTER 7
STRUCTURE SYSTEMS
AND COMPONENTS

7-1. General.

This chapter provides acceptance criteria applicable to structural systems for the design of military buildings. Advantages and disadvantages of each system are discussed, and pertinent detailing provisions are provided and illustrated with typical details. Alternative structural systems, other than those described in this chapter, should not be used in the design of military buildings without specific approval from the proponent agency.

a. Design for Life Safety. As indicated in Chapter 4, all buildings will be designed to protect life safety (Performance Objective 1A). Following the selection of the appropriate Seismic Use Group, Seismic Design Category, and analytical procedures, the design of the building is performed in accordance with the provisions of FEMA 302. Table 5.2.2 of "Design Coefficients and Factors of Basic Seismic-Force-Resisting Systems" is reproduced in this document as Table 7-1 for ease of reference.

b. Enhanced Performance Objectives. Chapter 4 prescribes the minimum analytical procedures for the enhanced performance objectives (Performance Objectives 2A, 2B, and 3B); the analytical procedures are discussed in Chapter 5; and numerical values for the acceptance criteria prescribed in Chapter 6 are provided in this chapter for various components of each structural systems. These values, modified from tables in FEMA 273, represent the current state of the art as defined by panels of experts. Future modification of these values should be expected as they are tested by analytical case studies and actual earthquakes. Alternative values, derived from test data in the literature or performed on a project-specific basis, may be used in lieu of the tabulated values, subject to the approval of the proponent agency.

(1) *m* factor tables. Tables of numerical acceptance criteria for linear procedures (*m* factors) are provided in this chapter for various deformation-controlled components and elements of structural systems. The columns in the left-hand side of the tables refer to the applicable condition of the component or element, and the columns on the right list the appropriate *m* factor for the following performance levels:

IO = immediate occupancy

SE = safe egress

LS = life safety

CP = collapse prevention.

The performance levels are defined in Table 4-3, and their physical significance is indicated in Figures 6-1 and 6-2.

(2) Nonlinear acceptance criteria. Tables of modeling parameters and numerical acceptance criteria for nonlinear procedures are provided for the same components and elements and in the same format as in the *m* factor tables for linear procedures.

TABLE 7-1
Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Bearing Wall Systems									
Ordinary steel braced frames	14 [k]	4	2	3½	NL	NL	160	160	160
Special reinforced concrete shear walls	9.3.2.4 [l]	5½	2½	5	NL	NL	160	160	160
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	4½	2½	4	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	9.3.2.2 [l]	2½	2½	2	NL	NL	NP	NP	NP
Ordinary plain concrete shear walls	9.3.2.1 [l]	1½	2½	1½	NL	NP	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	4	2½	3½	NL	NL	160	160	160
Intermediate reinforced masonry shear walls	11.11.4 [l]	3½	2½	3	NL	NL	NP	NP	NP
Ordinary reinforced masonry shear walls	11.11.3 [l]	2	2½	1¾	NL	NP	NP	NP	NP
Detailed plain masonry shear walls	11.11.2 [l]	2	2½	1¾	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	11.11.1 [l]	1½	2½	1¼	NL	NP	NP	NP	NP
Light frame walls with shear panels	3[n] 12.3.4.[l] 12.4 [l]	6½	3	4	NL	NL	160	160	100
Building Frame Systems									
Steel eccentrically braced frames, moment resisting, connections at columns away from links	15 [k]	8	2	4	NL	NL	160	160	100
Steel eccentrically braced frames, nonmoment resisting, connections at columns away from links	15 [k]	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	13 [k]	6	2	5	NL	NL	160	160	100
Ordinary steel concentrically braced frames	14 [k]	5	2	4½	NL	NL	160	100	100
Special reinforced concrete shear walls	9.3.2.4 [l]	6	2½	5	NL	NL	160	160	100
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	5	2½	4½	NL	NL	NP	NP	NP
Detailed plain concrete shear walls	9.3.2.2 [l]	3	2½	2½	NL	NL	NP	NP	NP
Ordinary plain concrete shear walls	9.3.2.1 [l]	2	2½	2	NL	NP	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Composite eccentrically braced frames	13 [m]	8	2	4	NL	NL	160	160	100
Composite concentrically braced frames	14 [m]	5	2	4½	NL	NL	160	160	100
Ordinary composite braced frames	12 [m]	3	2	3	NL	NL	NP	NP	NP
Composite steel plate shear walls	17 [m]	6½	2½	5½	NL	NL	160	160	100
Special composite reinforced concrete shear walls with steel elements	16 [m]	6	2½	5	NL	NL	160	160	100
Ordinary composite reinforced concrete shear walls with steel elements	15 [m]	5	2½	4½	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	5	2½	4	NL	NL	160	160	100
Intermediate reinforced masonry shear walls	11.11.4 [l]	4½	2½	4	NL	NL	160	160	100
Ordinary reinforced masonry shear walls	11.11.3 [l]	2½	2½	2¼	NL	NL	NP	NP	NP
Detailed plain masonry shear walls	11.11.2 [l]	2½	2 ½	2¼	NL	160	NP	NP	NP
Ordinary plain masonry shear walls	11.11.1 [l]	1½	2½	1¼	NL	NL	NP	NP	NP
Light frame walls with shear panels	12.3.4 [l] 12.4 [l]	7	2½	4½	NL	NL	160	160	160
Moment Resisting Frame Systems									
Special steel moment frames	9[k]	8	3	5 ½	NL	NL	NL	NL	NL
Special steel truss moment frames	12[k]	7	3	5 ½	NL	NL	160	100	NP
Intermediate steel moment frames	10[k]	6	3	5	NL	NL	160	100	NP[l]
Ordinary steel moment frames	11[k]	4	3	3½	NL	NL	35[l]	NP[ij]	NP[ij]
Special reinforced concrete moment frames	9.3.1.3[l]	8	3	5½	NL	NL	NL	NL	NL
Intermediate reinforced concrete moment frames	9.3.1.2[l]	5	3	4½	NL	NL	NP	NP	NP
Ordinary reinforced concrete moment frames	9.3.1.1[l]	3	3	2½	NL [a]	NP	NP	NP	NP
Special composite moment frames	9 [m]	8	3	5½	NL	NL	NL	NL	NL
Intermediate composite moment frames	10 [m]	5	3	4½	NL	NL	NP	NP	NP
Composite partially restrained moment	8 [m]	6	3	5½	160	160	100	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
frames									
Ordinary composite moment frames	11 [m]	3	3	4	NL	NP	NP	NP	NP
Masonry wall frames		5½	3	5	NL	NL	160	160	100
Dual Systems with Special Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces									
Steel eccentrically braced frames, moment resisting connections at columns away from links	15 [k]	8	2½	4	NL	NL	NL	NL	NL
Steel eccentrically braced frames, non-moment resisting connections, at columns away from links	15 [k]	7	2½	4	NL	NL	NL	NL	NL
Special steel concentrically braced frames	13 [k]	8	2½	6½	NL	NL	NL	NL	NL
Ordinary steel concentrically braced frames	14 [k]	6	2½	5	NL	NL	NL	NL	NL
Special reinforced concrete shear walls	9.3.2.4 [l]	8	2½	6½	NL	NL	NL	NL	NL
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	7	2½	6	NL	NL	NP	NP	NP
Composite eccentrically braced frames	13 [m]	8	2½	4	NL	NL	NL	NL	NL
Composite concentrically braced frames	14 [m]	6	2½	5	NL	NL	NL	NL	NL
Composite steel plate shear walls	17 [m]	8	3	6½	NL	NL	NL	NL	NL
Special composite reinforced concrete shear walls with steel elements	16 [m]	8	3	6½	NL	NL	NL	NL	NL
Ordinary composite reinforced concrete shear walls with steel elements	15 [m]	7	3	6½	NL	NL	NP	NP	NP
Special reinforced masonry shear walls	11.11.5 [l]	7	3	6½	NL	NL	NL	NL	NL
Intermediate reinforced masonry shear walls	11.11.4 [l]	6½	3	5½	NL	NL	NL	NP	NP
Dual Systems with Intermediate Moment Frames Capable of Resisting at Least 25% of Prescribed Seismic Forces									
Special steel concentrically braced frames [f]	13 [k]	6	2½	5	NL	NL	160	100	NP
Ordinary steel concentrically braced frames [f]	14 [k]	5	2½	4½	NL	NL	160	100	NP
Special reinforced concrete shear walls	9.3.2.4 [l]	6	2½	5	NL	NL	160	100	100
Ordinary reinforced concrete shear walls	9.3.2.3 [l]	5½	2½	4½	NL	NL	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

Basic Seismic-Force-Resisting System	Detailing Reference	Response Modification Co-Efficient, R^a	System Overstrength Factor, Ω_o^g	Deflection Amplification Factor, C_d^b	System Limitations and Building Height Limitations (ft) by Seismic Design Category[e]				
					B	C	D[d]	E[e]	F[f]
Ordinary reinforced masonry shear walls	11.11.3[l]	3	3	2½	NL	160	NP	NP	NP
Intermediate reinforced masonry shear walls	11.11.4[l]	5	3	4½	NL	NL	160	NP	NP
Composite concentrically braced frames	14[m]	5	2½	4½	NL	NL	160	100	NP
Ordinary composite braced frames	12[m]	4	2½	3	NL	NL	NP	NP	NP
Ordinary composite reinforced concrete shear walls with steel elements	16[m]	5	3	4½	NL	NL	NP	NP	NP
Inverted Pendulum Systems									
Special steel moment frames	9[k]	2½	2	2½	NL	NL	NL	NL	NL
Ordinary steel moment frames	11[k]	1¼	2	2½	NL	NL	NP	NP	NP
Special reinforced concrete moment frames	9.3.1.3[l]	2½	2	1¼	NL	NL	NL	NL	NL
Structural Steel Systems Not Specifically Detailed for Seismic Resistance		3	3	3	NL	NL	NP	NP	NP

TABLE 7-1. (cont'd)
Design Coefficients and Factors for Basic Seismic-Force-Resisting-Systems

NOTES FOR TABLE 7-1

[a] Response modification coefficient, *R*, for use throughout the Provisions of FEMA 302.

[b] Deflection amplification factor, C_d for use in Section 5.3.7.1 and 5.3.7.2. of FEMA 302.

[c] NL – not limited; and NP = not permitted. If using metric units, 100 feet approximately equals 30 m and 160 feet approximately equals 50 m.

[d] See Section 5.2.2.4.1 of FEMA 302 for a description of building systems limited to buildings with a height of 240 feet (70 m) or less.

[e] See Section 5.2.2.5 of FEMA 302 for building systems limited to buildings with a height of 160 feet (50 m) or less.

[f] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame in Seismic Design Categories B and C.

[g] The tabulated value of the overstrength factor, Ω_0, may be reduced by subtracting 1/2 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

[h] Ordinary moment frames of reinforced concrete are not permitted as a part of the seismic-force-resisting system in Seismic Design Category B structures founded on Site Class E or F soils (see Section 9.5.2 of FEMA 302).

[i] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 60 feet (18m) when the moment joints of field connections are constructed of bolted end plates, and the dead load of the roof does not exceed 15 psf (103 kPa).

[j] Steel ordinary moment frames are permitted in buildings up to a height of 35 feet (11mm) where the dead load of the walls, floors, and roofs does not exceed 15 psf (103 kPa).

[k] Refers to Section in Part I of AISC Seismic Provisions.

[l] Refers to Section in FEMA 302.

[m] Refers to Section in Part II of AISC Seismic Provisions.

[n] Refers to Chapter 3, paragraph 3 in TI 809-07.

As in the *m* factor tables, the columns on the left refer to the applicable condition of the component or element. The next three columns provide the appropriate modeling parameters or limit states for the various performance levels, as indicated in Figure 6-2c. The four columns on the right-hand side provide the appropriate values for the performance levels, as defined in Paragraph (1) above.

(3) Symbols and notations contained in the above tables are defined in Appendix B.

(4) Expected and lower-bound strengths. Default values for the determination of expected strength, Q_{CE}, for deformation-controlled components, and the lower-bound strength, Q_{CL}, for force-controlled components, are provided in Paragraph 6-3a(3). Specific exception to the default values is provided for some of the systems described in this chapter.

7-2. Shear Walls.

a. General.

(1) Function. Shear walls are vertical elements in the lateral-force-resisting system that transmit lateral forces from the diaphragm above to the diaphragm below, or to the foundation. Shear walls may also be bearing walls in the gravity-load system, or they may be components in a dual system framed so as to resist only lateral loads.

(2) Shear wall types. General discussions of shear walls are presented in Paragraphs 7-2b through 7-2e. Details of reinforced concrete shear walls are covered in Paragraph 7-2f, precast concrete shear walls in 7-2g, masonry shear walls in 7-2h, wood-stud shear walls in 7-2i, and steel stud shear walls in 7-2j.

(3) Revisions to ACI 318. Various revisions to Chapter 21 of ACI 318 have been approved, but have not yet been published (September 1998). Many of these revisions have been incorporated in Chapter 6 of FEMA 302 as modifications to the referenced provisions of ACI 318. The following provisions pertaining to mechanical and welded splices of reinforcement are not included in FEMA 302, but have been approved as revisions to ACI 318, and are incorporated as provisions required by this document:

(a) Delete Sections 21.2.6, 21.2.6.1, and 21.2.6.2 of ACI 318.

(b) Add the following new sections to ACI 318:

"21.2.6 - Mechanical splices

21.2.6.1 – Mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows:

(1) Type 1 mechanical splices shall conform to 12.14.3.4.

(2) Type 2 mechanical splices shall conform to 12.14.3.4 and shall develop the specified tensile strength of the spliced bar.

21.2.6.2 – Type 1 mechanical splices shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Type 2 mechanical splices shall be permitted to be used at any location.

21.2.7 – Welded splices

21.2.7.1 – Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.3 and shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

21.2.7.2 – Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design shall not be permitted."

b. Design Forces. Walls may be subjected to both vertical (gravity) and horizontal (wind or earthquake) forces. The horizontal forces are both in-plane and out-of-plane. When considered under their in-plane loads, walls are called shear walls; when considered under their out-of-plane loads, they are called normal walls. Walls will be designed to withstand all vertical loads and horizontal forces, both parallel to and normal to the flat surface, with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall, whether or not intended as part of the lateral-force-resisting system, is subjected to lateral forces unless it is isolated on three sides (both ends and top), in which case it is classified as nonstructural. Any wall that is not isolated will participate in shear resistance to horizontal forces parallel to the wall, since it tends to deform under stress when the surrounding framework deforms.

c. Wall Components. Reinforced concrete and reinforced masonry shear walls are seldom simple walls. Whenever a wall has doors, windows, or other openings, the wall must be considered as an assemblage of relatively flexible components (column segments and wall piers), and relatively stiff elements (wall segments).

(1) Column segments. A column segment is a vertical member whose height exceeds three times its thickness, and whose width is less than two and one-half times its thickness. Its load is usually predominantly axial. Although it may contribute little to the lateral-force resistance of the shear wall, its rigidity must be considered. When a column is built integral with a wall, the portion of the column that projects from the face of the wall is called a pilaster. Column segments shall be designed according to ACI 318 for concrete and ACI 530 for masonry.

(2) Wall piers. A wall pier is a segment of a wall whose horizontal length is between two and one-half and six times its thickness, and whose clear height is at least two times its horizontal length.

(3) Wall segments. Wall segments are components that are longer than wall piers. They are the primary lateral-load-resisting components in the shear wall.

d. In-Plane Effects. Horizontal forces at any floor or roof level are generally transferred to the ground (foundation) by using the strength and rigidity of shear walls (and partitions). A shear wall may be considered analogous to a cantilever plate girder standing on end in a vertical plane, where the wall performs the function of a plate girder web, the pilasters or floor diaphragms function as web stiffeners, and the integral reinforcement of the vertical boundaries functions as flanges. Axial, flexural, and shear forces must be considered in the design of shear walls. The tensile forces on shear wall elements resulting from the combination of seismic uplift forces and seismic overturning moments must be resisted by anchorage into the foundation medium unless the uplift can be counteracted by gravity loads (e.g., 0.90 of dead load) mobilized from neighboring elements. A shear wall may be constructed of materials such as concrete, wood, unit masonry, or metal in various forms. Design procedures for such materials as cast-in-place reinforced concrete and reinforced unit masonry are well known, and present no problem to the designer once the loading and reaction system is determined. Other materials frequently used to support vertical loads from floors and roofs have well- established vertical- load- carrying characteristics, but have required tests to demonstrate their ability to resist lateral forces. Various types of wood sheathing and metal siding fall into this category. Where a shear wall is made up of units such as plywood, gypsum, wallboard, tilt-up concrete units, or metal panel units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another, and to the supporting members.

(1) Rigidity analysis. For a building with rigid diaphragms, there is a torsional moment, and a rigidity analysis is required. It is necessary to make a logical and consistent distribution of story shears to each wall. An exact determination of wall rigidities is very difficult, but is not necessary, because only *relative* rigidities are needed. Approximate methods in which the deflections of portions of walls are combined usually are adequate.

(a) Wall deflections. The rigidity of a wall is usually defined as the force required to cause a unit deflection. Rigidity is expressed in kips per inch. The deflection of a concrete shear wall is the sum of the shear and flexural deflections (see Figure 7-1). In the case of a solid wall with no openings, the computations of deflection are quite simple; however, where the shear wall has openings, as for doors and windows, the computations for deflection and rigidity are much more complex. An exact analysis, considering angular rotation of elements, rib shortening, etc., is very time-consuming. For this reason, several short-cut approximate methods have been developed. These do not always give consistent or satisfactory results. A conservative approach and judgment must be used.

(b) Deflection charts. The calculation of deflections is facilitated by the use of the deflection charts. See Figure 7-4 for fixed-ended corner and rectangular piers. Curves 5 and 6 are for cantilever corner and rectangular piers. The corner pier curves are for the special case where the moment of inertia,

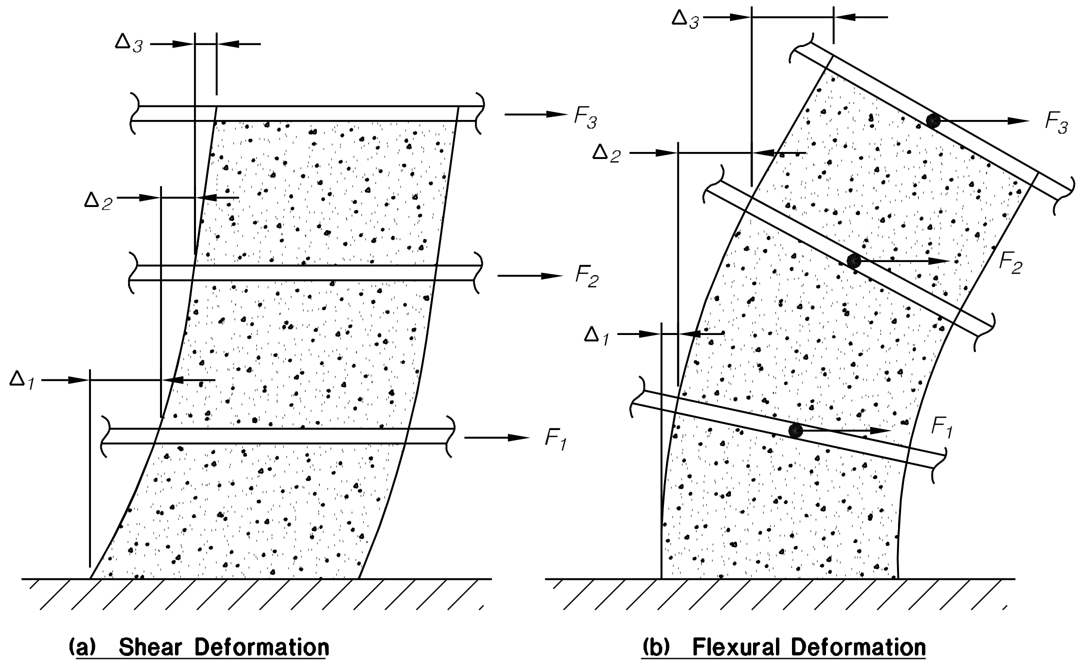


Figure 7-1 Shear wall deformation.

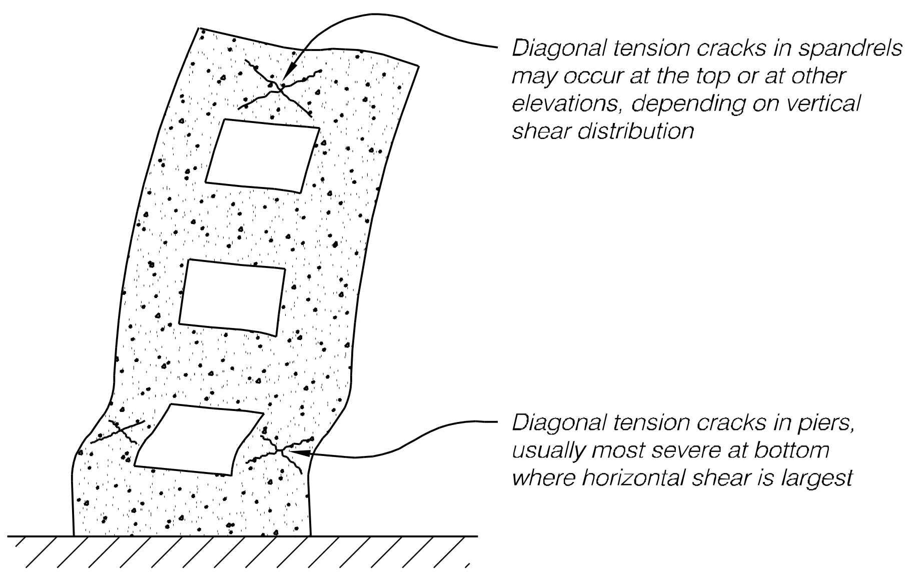


Figure 7-2 Deformation of shear wall with openings.

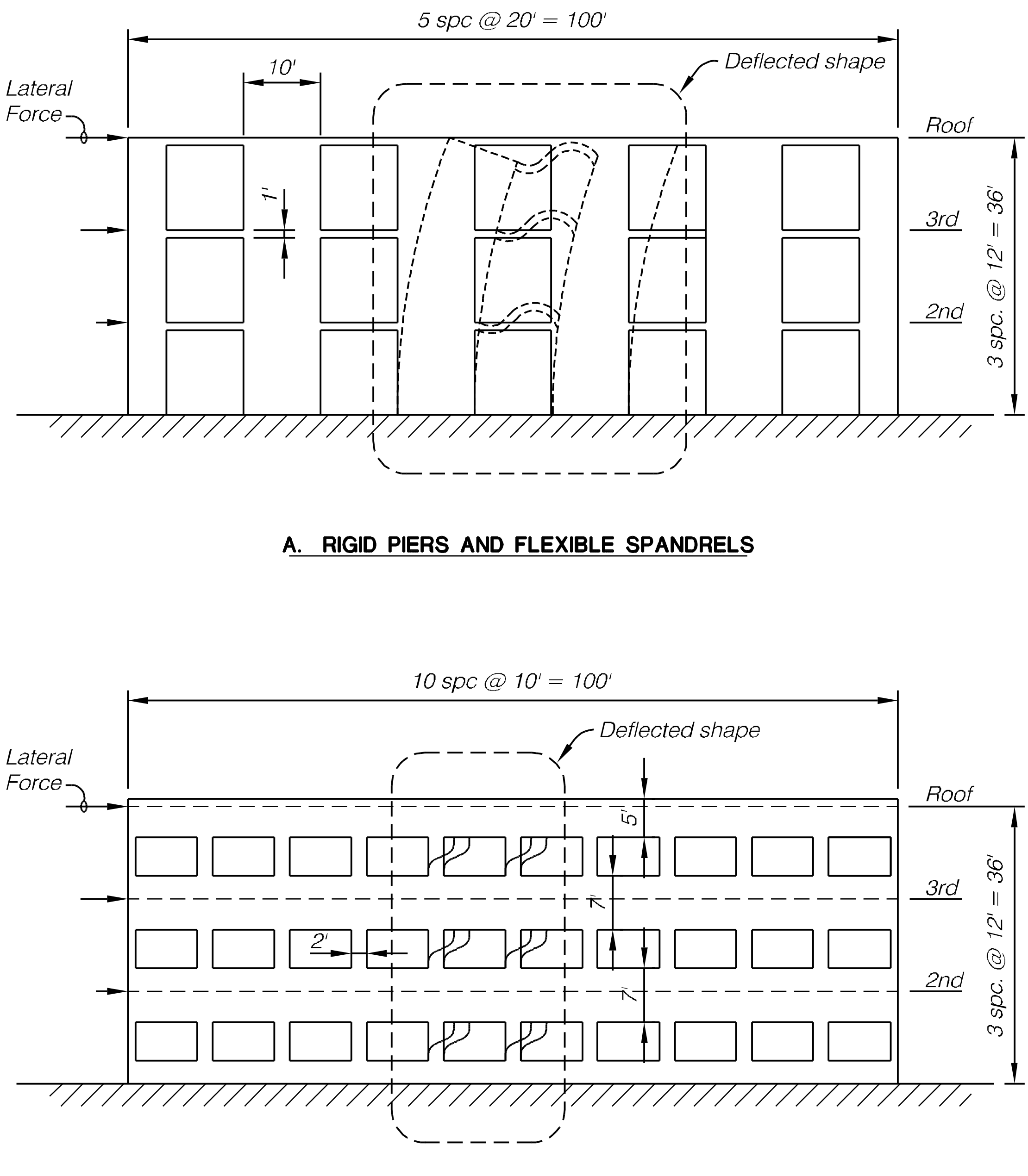
I, of the corner pier is 1.5 times that of a rectangular pier; for other *I*-values, the bending portion of the deflection would be proportional. The deflections shown on the charts are for a horizontal load, *P*, of 1,000,000 pounds. The deflections shown on the charts are reasonably accurate. The formulas written on the curves can be used to check the results; however, the charts will give no better results than the assumptions made in the shear wall analysis. For instance, the point of contraflexure of a vertical pier may not be in the center of the pier height. In some cases, the point of contraflexure may be selected by judgment and an interpolation made between the cantilever and fixed conditions.

(c) Foundation effects. The rotation at the foundation can greatly influence the overall rigidity of a shear wall because of the very rigid nature of the shear wall itself; however, the rotational influence on relative rigidities of walls for purposes of horizontal force distribution may not be as significant. Considering the complexities of soil behavior, a quantitative evaluation of the foundation rotation is generally not practical, but a qualitative evaluation will be provided.

(d) Framework effects. The relative rigidity of concrete or unit masonry walls with nominal openings is usually much greater than that of the building framework; therefore, the walls tend to resist essentially all or a major part of the lateral force.

(2) Effect of openings. The effect of openings on the ability of shear walls to resist lateral forces must be considered. If openings are very small, their effect on the overall state of stress in a shear wall is minor. Large openings have a more pronounced effect, and if large enough, result in a system in which typical frame action predominates. Openings commonly occur in regularly spaced vertical rows throughout the height of the wall, and the connection between the wall sections is provided by either connecting beams (or spandrels) which form a part of the wall, or floor slabs, or a combination of both. If the openings do not line up vertically and/or horizontally, the complexity of the analysis is greatly increased. In most cases, a rigorous analysis of a wall with openings is not required. “Strut and Tie” procedures that depict shear walls as consisting of compression struts and tension ties are useful tools for the evaluation of shear walls with openings (see Paulay and Priestley, 1992). In the design of a wall with openings, the deformations must be visualized in order to establish some approximate method for analyzing the stress distribution to the wall. Figures 7-3 and 7-4 give some visual descriptions of such deformations. The major points that must be considered are the lengthening and shortening of the extreme sides (boundaries) due to deep beam action, the stress concentration at the corner junctions of the horizontal and vertical components between openings, and the shear and diagonal tension in both the horizontal and vertical components.

(a) Relative rigidities of piers and spandrels. The ease of methods of analysis for walls with openings is greatly dependent on the relative rigidities of the piers and the spandrels, as well as the general geometry of the building. Figure 7-3 shows two extreme examples of relative rigidities of exterior walls of a building. In Figure 7-3A, the piers are very rigid and the spandrels are very flexible. Assuming a rigid base, the shear walls act as vertical



A. RIGID PIERS AND FLEXIBLE SPANDRELS

B. FLEXIBLE PIERS AND RIGID SPANDRELS

Figure 7-3 Relative rigidities of piers and spandrels.

1 foot = 0.3 meter

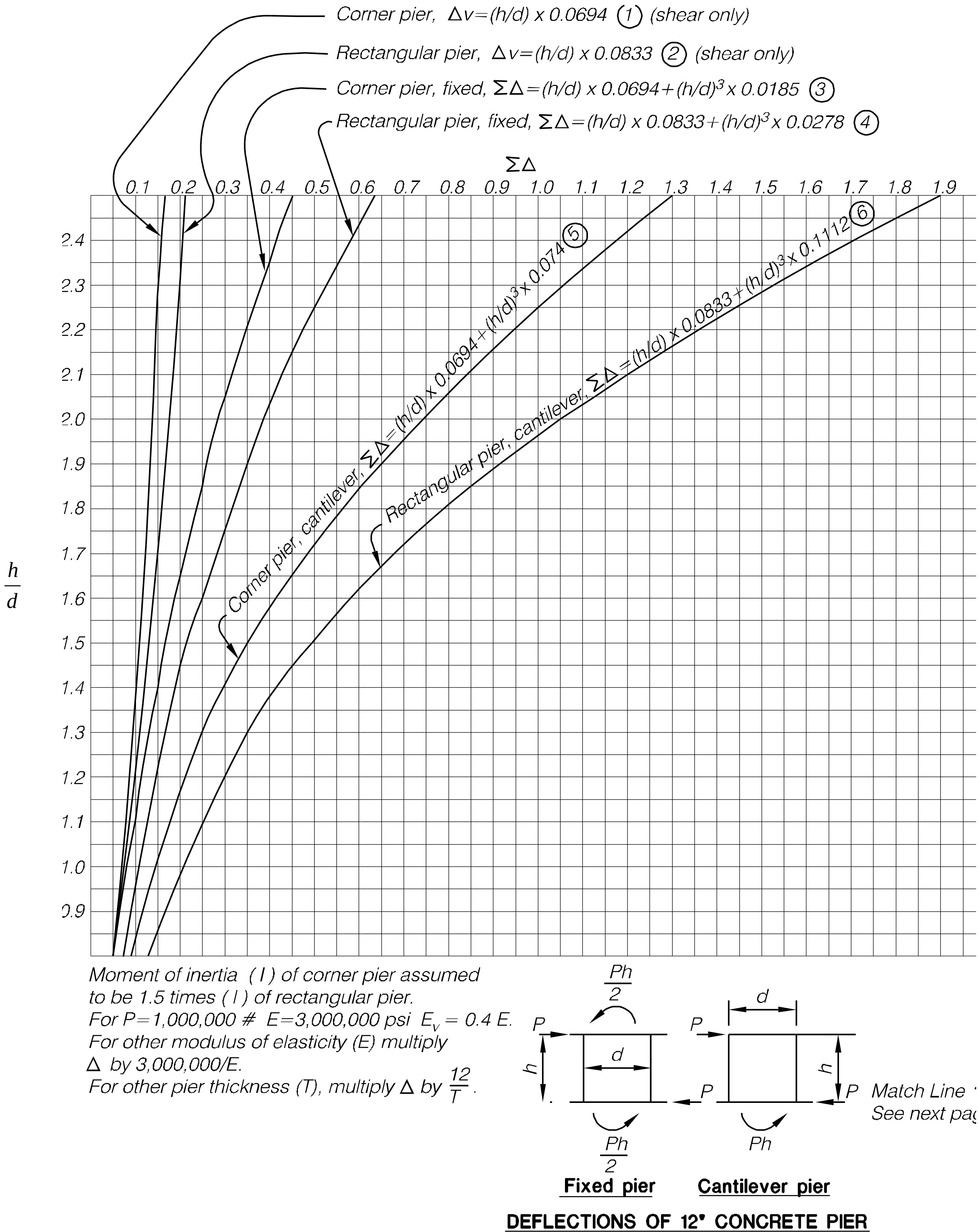
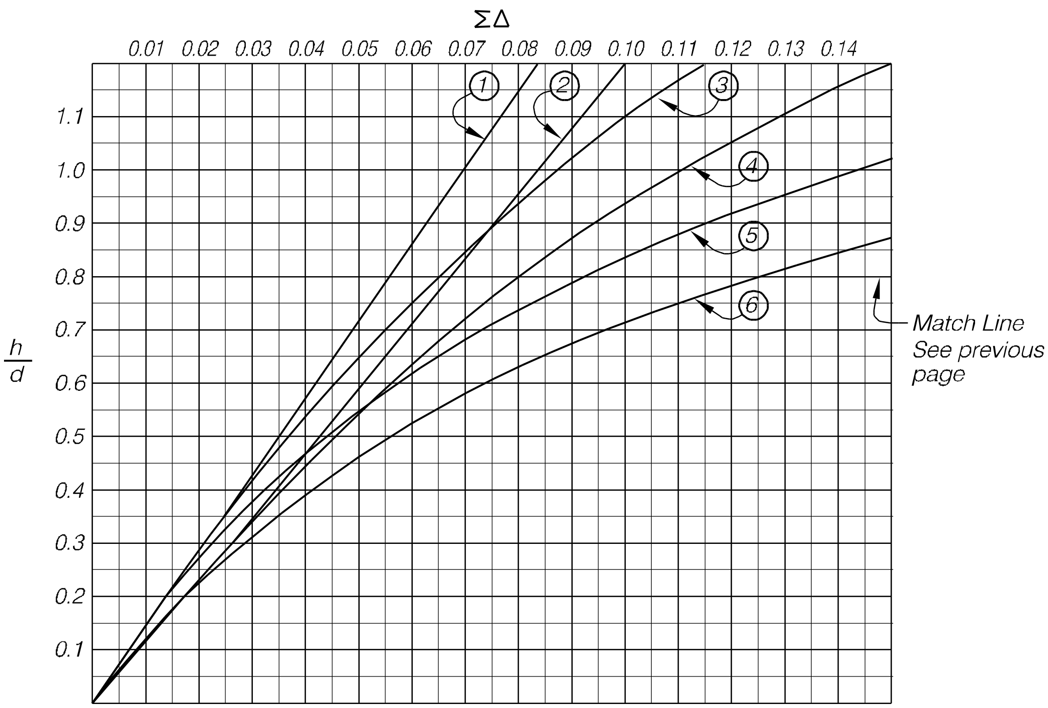


Figure 7-4 Design curves for masonry and concrete shear walls.



Metric equivalents:

Curves based on:

T = 305mm
P = 4448 MN
E = 20,685 Mpa

For other modulus of elasticity, (E), multiply Δ by 20,685/E.

For other pier thickness, (T), multiply Δ by 305/T.

Figure 7-4 Design curves - continued.

cantilevers. When a lateral force is applied, the spandrels act as struts that flexurally deform to be compatible with the deformation of the cantilever piers. It is relatively simple to determine the forces on the cantilever piers by ignoring the deformation characteristics of the spandrels. The spandrels are then designed to be compatible with the pier deformations. In Figure 7-3B, the piers are relatively flexible compared with the spandrels. The spandrels are assumed to be infinitely rigid, and the piers are analyzed as fixed-ended columns. The spandrels are then designed for the forces induced by the columns. The overall wall system is also analyzed for overturning forces that induce axial forces into the columns. The calculations of relative rigidities for both cases shown in Figure 7-3 can be aided by the charts in Figure 7-4. For cases of relative spandrel and pier rigidities other than those shown, the analysis and design become more complex.

(3) Methods of analysis. Approximate methods for analyzing walls with openings are generally acceptable. For the extreme cases shown in Figure 7-3, the procedure is straightforward. For other cases, a variety of assumptions may be used to determine the most critical loads on various elements, thus resulting in a conservative design. (Note: In some cases, a few additional reinforcing bars, at little additional cost, can greatly increase the strength of shear walls with openings.) When, however, the reinforcement requirements or the resulting stresses of this approach appear excessively large, the strut and tie procedure indicated in paragraph 7-2d (2) or a more rigorous analysis may be justified.

(4) Coupled shear walls. When two or more shear walls in one plane are linked together by coupling beams, interactive forces are transmitted to the shear walls by the beams. In addition to these axial forces, the beams develop moments and shears that contribute to the walls resisting overturning. The magnitude of the resisting beam bending moments and vertical shears is dependent on the relative stiffnesses of the walls and the coupling beams. It should be noted that the foundation itself functions as a coupling beam. Accurate determination of the resisting forces can be complex; therefore, approximate methods are generally used. One method may be used for calculating the axial forces, and another method may be used for calculating bending moments and shears to ensure that the structural elements are not underdesigned.

(5) Construction joints and dowels. The contact faces of shear wall construction joints have exhibited slippage and related drift damage in past earthquakes. Consideration must be given to the location and details of construction joints, which must be clean and roughened. Shear friction reinforcement may be utilized in accordance with ACI 318. For this procedure, a coefficient of friction of 0.6 is suggested for seismic effects.

e. Out-of-Plane Effects.

(1) Lateral forces. Walls and partitions must safely resist horizontal seismic forces normal to their flat surface (Figure 7-5, part *a*). At the same time, they must resist moments and shears induced by relative deflections of the diaphragms above and below (Figure 7-5, part *b*). The normal force on a wall is a function of its weight. Equations for the determination of the force are provided in Paragraph 10-1b(1); however, wind forces, other forces, or interstory drift will frequently govern the design.

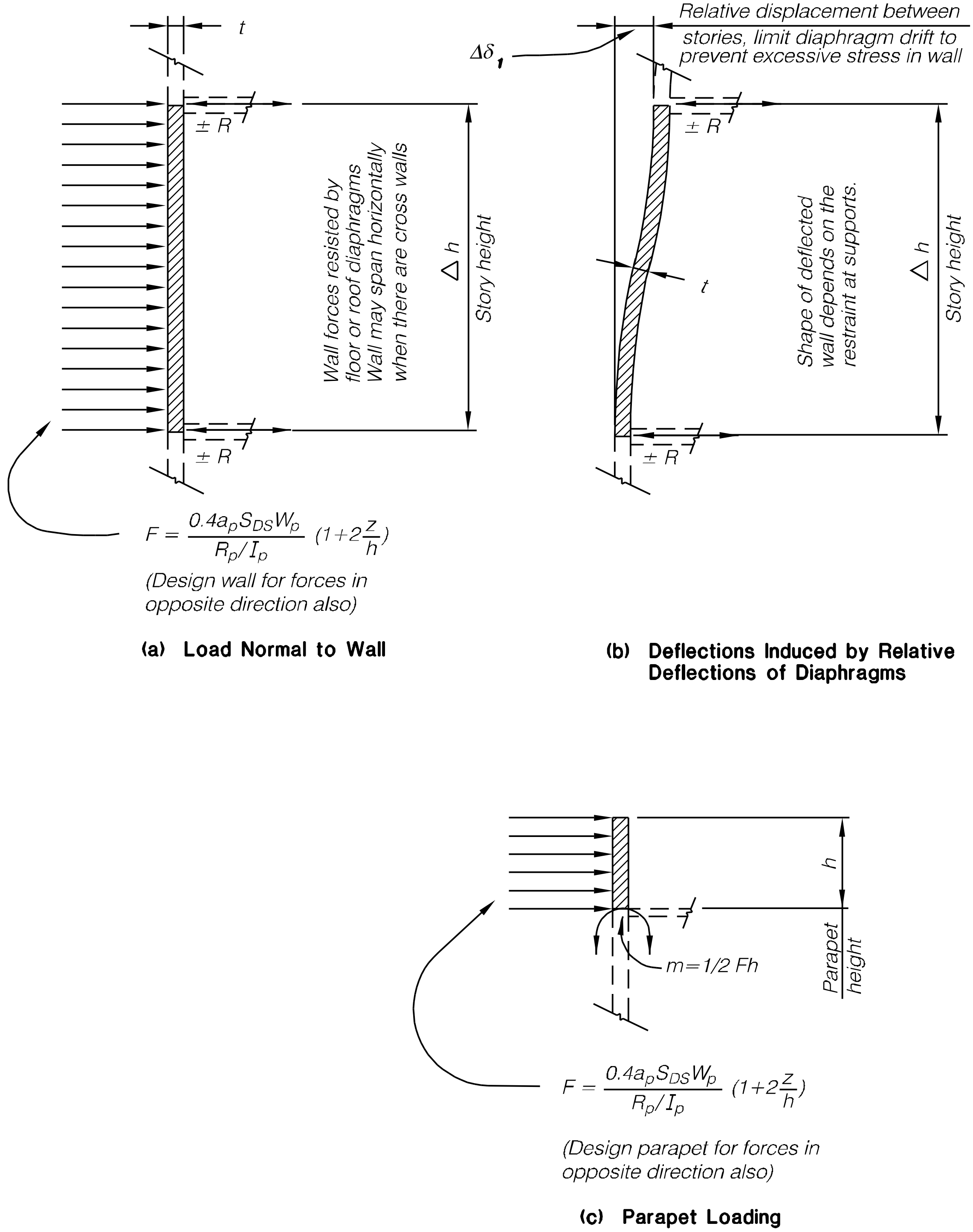


Figure 7-5 Out-of-plane effects.

(For cantilevered walls, see Paragraph 3 below.) The design force will be applied to the wall in both inward and outward directions.

(2) Wall behavior. Walls distribute normal forces vertically to the horizontal resisting elements above or below. They may also distribute normal forces to frames, or other walls or frames. A wall may be either continuous or discontinuous across its supports.

(3) Cantilevered walls. Where walls, such as parapets, are cantilevered, the anchorage for reaction and cantilever moment is required to be fully developed (Figure 7-5, part *c*). Where a parapet wall is anchored to a concrete roof slab and is not a continuation of a wall below, the roof slab will be designed for the cantilever moment. Where the parapet is a continuation of a wall below, the cantilever moment will be divided between the concrete slab and the wall below in proportion to their relative stiffnesses. Where the parapet is an extension of a wall below and is anchored to a roof or floor of wood, metal deck, or other similar materials, the moment at the base of the parapet will be developed into the wall below. In this case, the anchorage force to the roof will be determined by the usual methods of analysis, assuming a pinned condition for the connection of the roof to the wall.

(4) Connections. Walls will be anchored to the structural frame or diaphragm by dowels, anchor bolts, or other approved methods to withstand the design forces, but in no case less than 200 pounds per linear foot. Dovetail anchors are inadequate for this purpose. Nonstructural partitions will be isolated from exterior walls and shear partitions so as to prevent buttress action, which would restrict shear walls from deflecting with the diaphragms. Isolated partitions will be braced to overhead construction or anchored to other isolated cross walls to ensure lateral stability under out-of-plane loading.

f. Cast-in-Place Concrete Shear Walls.

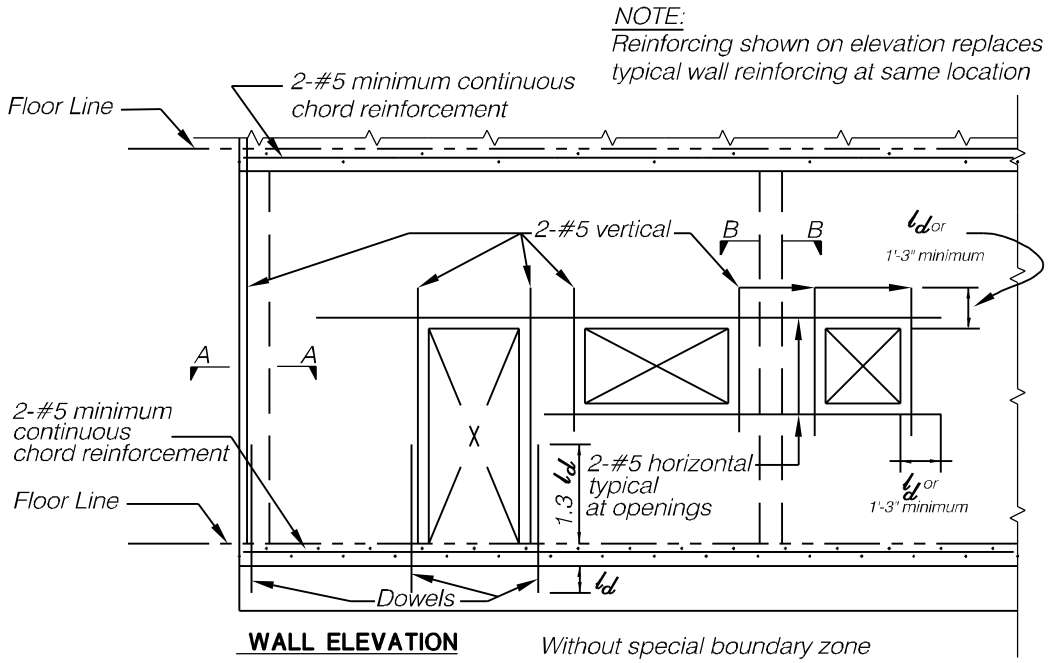
(1) General requirements. Reinforced concrete shear walls shall comply with the following provisions.

(a) Seismic Design Categories A and B. Shear walls may be of any type permitted by ACI 318 and FEMA 302.

(b) Seismic Design Category C. Shear walls may be detailed plain concrete shear walls, ordinary reinforced concrete shear walls, or special reinforced concrete shear walls as prescribed in Chapter 9 of FEMA 302.

(c) Seismic Design Categories D, E, and F. Shear walls shall be special reinforced concrete shear walls in accordance with Section 9.3.2.4 of FEMA 302.

(2) General design criteria. The criteria used to design reinforced concrete shear walls will be ACI 318 requirements for “structural walls,” as modified by the provisions given in Chapter 9 of FEMA 302. For tilt-up and other precast concrete shear walls, refer to Paragraph 7-2g. For details of reinforcement, see Figure 7-6 and 7-7.



NOTE:
All reinforcement shown is typical unless additional or larger bars are required by seismic design.

1 inch = 25mm
#4 bar ≈ 10M bar
#5 bar = 15M bar

Table	
MINIMUM CONCRETE WALL REINFORCING	
Wall Thickness	Vertical and horizontal Reinforcing *
8"	#4 at 18" o.c. both faces
9"	#4 at 18" o.c. both faces
10"	#4 at 16" o.c. both faces
12"	#4 at 13" o.c. both faces

* Max. spacing of bars = d/3 where d is dimension of the wall pier parallel to shear force

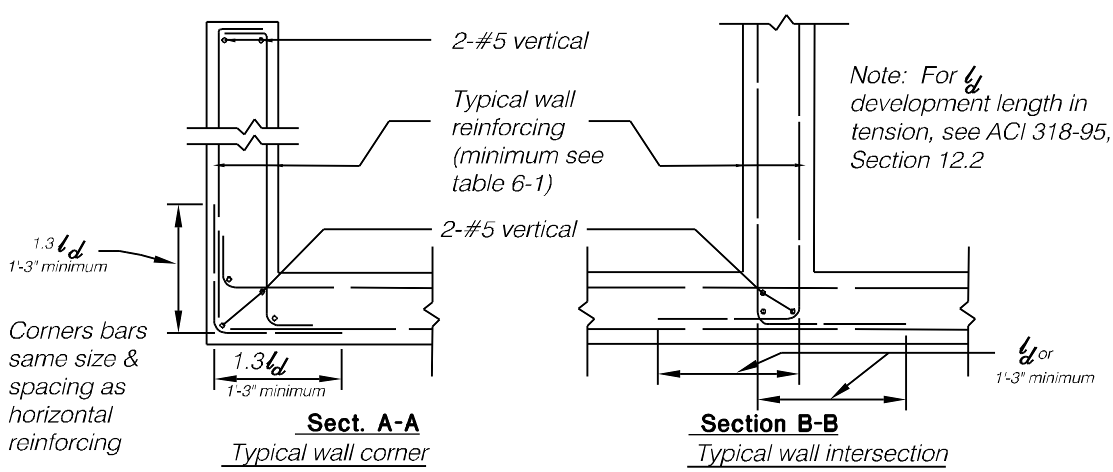


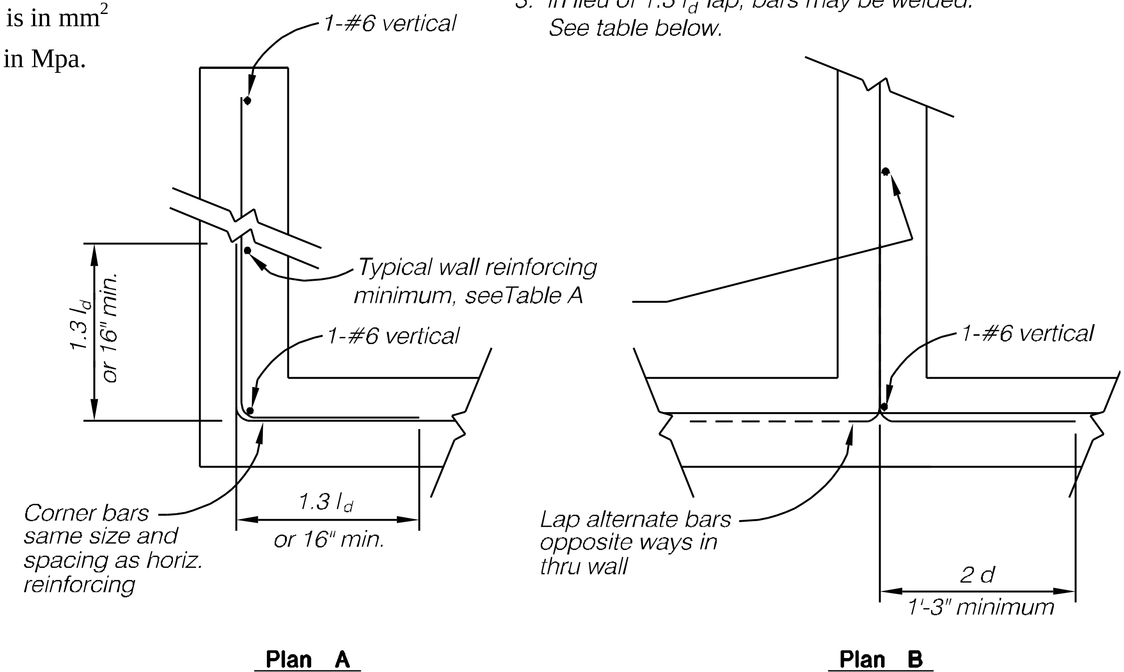
Figure 7-6 Minimum concrete shear wall reinforcement (two curtains)

1 inch = 25mm
#4 bar ≈ 10M bar
#5 bar = 15M bar
#6 bar ≈ 20M bar
#8 bar ≈ 25M bar

$$2A_{cv}\sqrt{f_c^{'}} = \frac{A_{cv}\sqrt{f_c^{'}}}{6}$$

where A_{cv} is in mm^2
and $f_c^{'}$ is in Mpa.

Note: 1. *Bars at jams, heads, and sills of openings will be 1 - #4 placed in same manner as indicated for two curtain reinf. or Figure 7-6.*
2. *All reinforcement shown is typical unless additional or larger bars are required by seismic design.*
3. *In lieu of 1.3 l_d lap, bars may be welded. See table below.*



TYPICAL WALL CORNER AND JAMS

TYPICAL WALL INTERSECTION

Table A

MINIMUM CONCRETE WALL REINFORCEMENT	
Wall Thickness	*Vertical and horizontal Reinforcing **
5"	#4 at 16" o.c. in center
6"	#4 at 13" o.c. in center
7"	#4 at 11" o.c. in center
8"	#4 at 10" o.c. in center

** Spacing of bars not to exceed d/3 where d is dimension of the wall pier parallel to shear force*

Table B

MINIMUM LENGTH OF STANDARD A.W.S. FLARE GROOVE WELDS TO DEVELOP LAPPED REINFORCING BARS	
Bar	*Welded length (each side)*
3	2"
4	2 1/2"
5	3"
6	3 1/2"
7	4"

Note: Special reinforced concrete shear walls shall have two curtains of reinforcement when the factored shear force exceeds 2 $A_{cv}\sqrt{f_c'}$

Figure 7-7 Minimum concrete shear wall reinforcement (one curtain).

(3) Boundary zone requirements for special reinforced concrete shear walls.

(a) Boundary zones are required for special reinforced concrete shear walls, except where the following conditions exist.

1. $P_u \le 0.10\, A_g\, f_c'$ for geometrically symmetrical wall sections $P_u \le 0.05\, A_g\, f_c'$ for geometrically unsymmetrical wall sections, and either

2. $M_u/V_u\, l_w \le 1.0$

or

3. $V_u \le 3A_{cv}\sqrt{f_c'}$ and $M_u/V_u\, l_w \le 3.0$

The metric equivalent is $v_u \le A_{cv}\sqrt{f_e'}\,/\,4$ where:

P_u = 1.2D+0.5L + E, kips (kN).

A_g = gross area of the wall, in^2 (mm^2).

M_u = required moment strength, kip-in (kN-m).

V_u = required shear strength, kips (kN).

l_w = horizontal length of wall, in (mm).

A_{cv} = net area of wall bounded by the web thickness and length of section in^2 (mm^2).

(b) Design and detailing of boundary zones shall be in accordance with the provisions of ACI 318, as modified by Section 9.1.1.13 of FEMA 302. Section 9.1.1.13 in FEMA 302 modifies the boundary zone provisions in Section 21.6.6 of ACI 318 with more explicit statements regarding when boundary zones are required; the design of boundary zones; and when the boundary zone is no longer required.

(c) Wall boundary elements may also occur in the building frame system and the dual system where the usual configuration is to place the shear walls within the bays between the frame columns. See Figure 7-8 for details of shear walls with boundary elements. Note that the vertical reinforcement in the boundary zones in Figure 7-8 is extended to be developed above and below the prescribed limit of the boundary zones.

(d) When boundaries are not required, special reinforced concrete shear walls shall comply with Section 21.6 of ACI 318, as modified by the applicable provisions of Section 9.1.1.13 of FEMA 302.

(4) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1.

(b) Modification factors, *m*, for enhanced performance objectives are provided in Table 7-2 for components controlled by flexure, and in Table 7-3 for components controlled by shear.

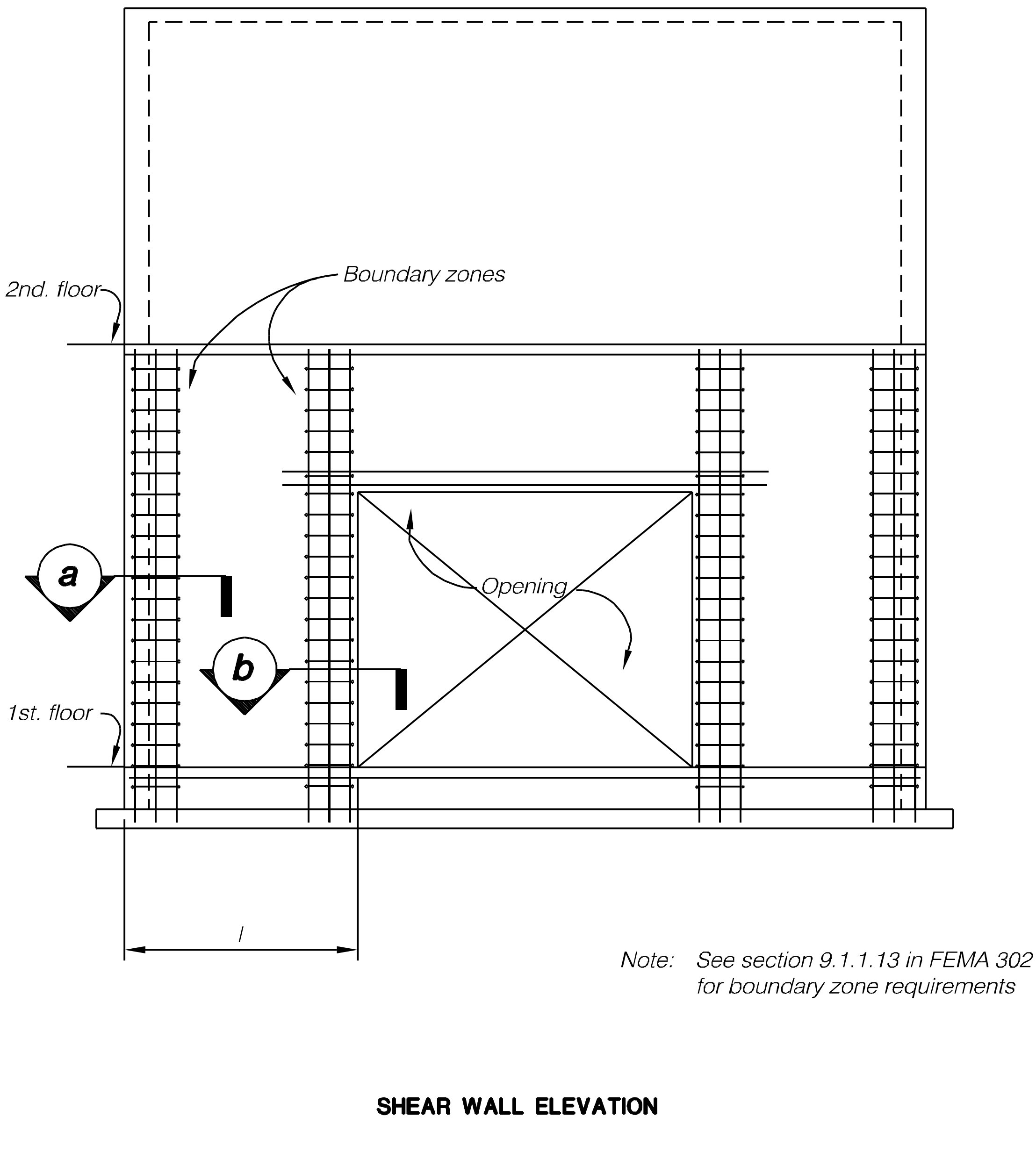
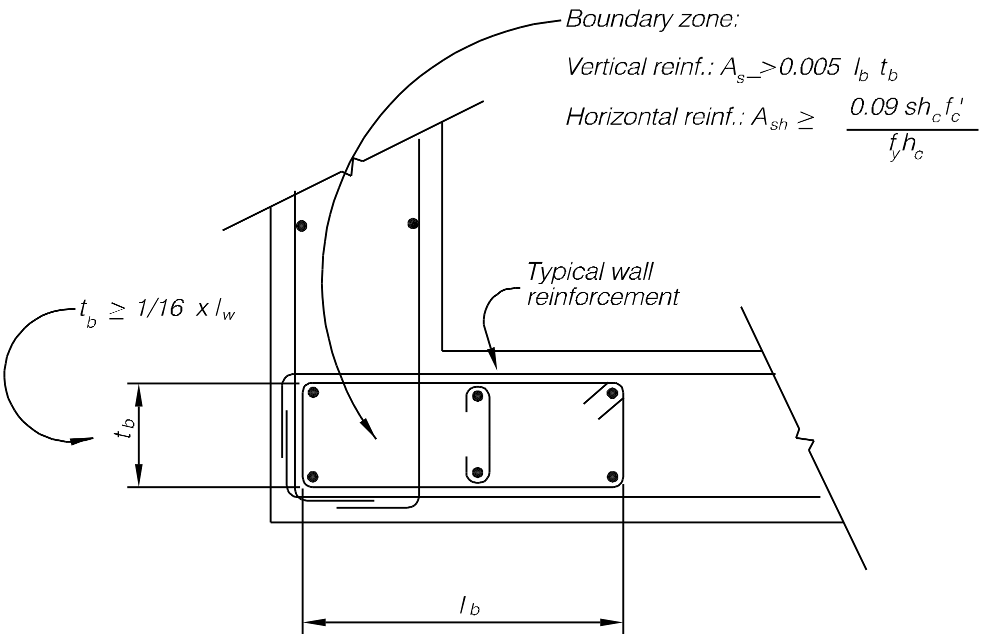
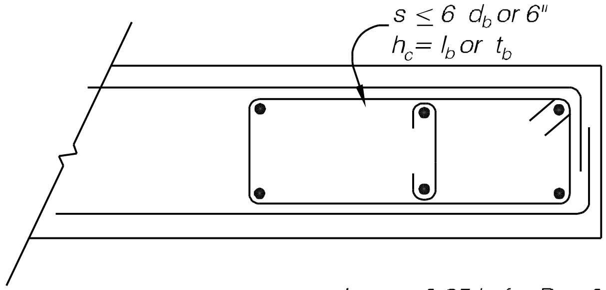


Figure 7-8 Boundary zones in a special reinforced concrete shear wall



Section A



$l_b = 0.25\ l_w$ for $P_u = 0.35\ P_o$
$=0.15\ l_w$ for $P_u = 0.15\ P_o$
$=$ 18 in (460 mm) min.

Section B

Figure 7-8 cont. Boundary zones in a special reinforced concrete shear wall

Conditions			m factors					
			Component Type					
			Primary				Secondary	
			Performance Level					
			IO	**SE**	**LS**	**CP**	**LS**	**CP**
i. Shear walls and wall segments								
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]						
≤ 0.1	≤ 3	Yes	2	3	4	6	6	8
≤ 0.1	≥ 6	Yes	2	2.5	3	4	4	6
≥ 0.25	≤ 3	Yes	1.5	2.3	3	4	4	6
≥ 0.25	≥ 6	Yes	1	1.5	2	2.5	2.5	4
≤ 0.1	≤ 3	No	2	2.3	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	1.8	2	2.5	2.5	4
≥ 0.25	≤ 3	No	1	1.3	1.5	2	2	3
≥ 0.25	≥ 6	No	1	1	1	1.5	1.5	2
ii. Columns supporting discontinuous shear walls								
Transverse reinforcement[2]								
Conforming			1	1.3	1.5	2	n.a.	n.a.
Nonconforming			1	1	1	1	n.a.	n.a.
iii. Shear wall coupling beams								
Longitudinal reinforcement and transverse reinforcement[3]		$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$						
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	3	4	6	6	9
		≥ 6	1.5	2.3	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	2.5	3.5	5	5	8
		≥ 6	1.2	1.5	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	3.5	5	7	7	10

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Table 7-2: Numeric Acceptance Criteria for Linear Procedures—Members Controlled by Flexure

Conditions		m factors					
		Component Type					
		Primary				Secondary	
		Performance Level					
		IO	SE	LS	CP	LS	CP
i. Shear walls and wall segments							
All shear walls and wall segments[1]		2	2	2	3	2	3
ii. Shear wall coupling beams							
Longitudinal reinforcement and transverse reinforcement[2]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$						
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	2.3	3	4	4	6
	≥ 6	1.2	1.6	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2	2.5	3	3	4
	≥ 6	1	1.1	1.2	1.5	1.5	2.5

1. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15\, A_g f_c'$, the longitudinal reinforcement must be symmetrical, and the maximum shear stress must be $\leq 6\sqrt{f_c'}$, otherwise the shear shall be considered to be a force-controlled action.

2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

Table 7-3: Numeric Acceptance Criteria for Linear Procedures—Members Controlled by Shear

(c) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-4 for members controlled by flexure, and in Table 7-5 for members controlled by shear.

(d) Expected strength, Q_{CE}, shall be determined using $1.25f_y$ for the contribution attributed to the reinforcement in flexure and $1.0\ f_y$ in shear.

(e) Lower-bound strength for force-controlled actions (e.g., length of splices, dowels, or embedments) shall be equal to the values provided in ACI 318 without a strength reduction factor, N.

g. Tilt-up and Other Precast Concrete Shear Walls.

(1) Analysis. Precast concrete shear walls shall be designed in accordance with Section 9.1.1.5 of FEMA 302, and shall emulate the behavior of monolithic reinforced concrete construction. Where tilt-up or precast concrete walls are used as shear walls, the analysis is similar to that for walls of cast-in-place concrete; however, in this case, the boundary conditions become critical, and the shears between precast and cast-in-place elements must be analyzed. Shears between two precast elements or between a precast element and a cast-in-place element may be developed by shear keys, dowels, or welded inserts. The contact joint itself is a cold joint, and will be given no shear or tension value.

(2) Joints. Precast concrete elements tend to be structurally separate, one element from another. In the case of precast wall construction, for instance, these could be a series of concrete elements tied together at top and bottom, but structurally separated from each other by vertical joints. Since all elements in a line are tied together at the top, they must have equal horizontal deflections; therefore, a horizontal force parallel to the line of units will be resisted by the individual elements in proportion to relative rigidities. Such elements may not have equal rigidities, since some may contain large openings or may be of different height-width ratios. Some elements may deflect primarily in shear, and others primarily in flexure. Where significant dissimilar deflections are found, the building elements tying the individual units together must be analyzed to determine their ability to resist or accept such deformations, including angular rotation, without losing their ability to function as ties or diaphragm chords or footings. Mechanical keys or sleeved dowels may be used to assist in eliminating differential movement of adjacent precast panels separated by control joints where appearance and weather-tightness are otherwise satisfactorily provided.

(3) Connectors for shear walls. Past earthquakes have shown that the performance of weld plates or other nonductile connectors has often been poor, and in many cases they have resulted in failures. These connectors have been weak links in the shear wall connection. It is important that the load-bearing shear walls be more stringently or conservatively designed, since any connector failure during an earthquake may result in progressive

Conditions			Plastic Hinge Rotation (radians)		Residual Strength Ratio	Acceptable Plastic Hinge Rotation (radians)					
						Component Type					
						Primary				Secondary	
						Performance Level					
			a	b	c	IO	SE	LS	CP	LS	CP
i. Shear walls and wall segments											
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]									
≤ 0.1	≤3	Yes	0.015	0.020	0.75	0.005	0.008	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.006	0.008	0.010	0.010	0.015
≥ 0.25	≤ 3	Yes	0.009	0.012	0.60	0.003	0.005	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.001	0.002	0.003	0.005	0.005	0.010
≤ 0.1	≤ 3	No	0.008	0.015	0.60	0.002	0.003	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.003	0.004	0.006	0.006	0.010
≥ 0.25	≤ 3	No	0.003	0.005	0.25	0.001	0.002	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.001	0.002	0.002	0.004
ii. Columns supporting discontinuous shear walls											
Transverse reinforcement[2]											
Conforming			0.010	0.015	0.20	0.003	0.005	0.007	0.010	n.a.	n.a.
Nonconforming			0.0	0.0	0.0	0.0	0.0	0.0	0.0	n.a.	n.a.
			Chord Rotation (radians)								
			d	e							
iii. Shear wall coupling beams											
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f'_c}}$										
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3		0.025	0.040	0.75	0.006	0.011	0.015	0.025	0.025	0.040
	≥ 6		0.015	0.030	0.50	0.005	0.008	0.010	0.015	0.015	0.030
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3		0.020	0.035	0.50	0.006	0.009	0.012	0.020	0.020	0.035
	≥ 6		0.010	0.025	0.25	0.005	0.007	0.008	0.010	0.010	0.025
Diagonal reinforcement	n.a.		0.030	0.050	0.80	0.006	0.012	0.018	0.030	0.030	0.050

1. Requirements for a confined boundary are the same as those given in *ACI 318-95*.
2. Requirements for conforming transverse reinforcement are: (a) closed stirrups over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of closed stirrups V_s ≥ required shear strength of column.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of beam.

Table 7-4: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures— Members Controlled by Flexure

Conditions		Drift Ratio (%), or Chord Rotation (radians)[1]		Residual Strength Ratio	Acceptable Drift (%) or Chord Rotation (radians)[1]					
					Component Type					
					Primary				Secondary	
					Performance Level					
		d	*e*	*c*	IO	SE	LS	CP	LS	CP
i. Shear walls and wall segments										
All shear walls and wall segments[2]		0.75	2.0	0.40	0.40	0.50	0.60	0.75	0.75	1.5
ii. Shear wall coupling beams										
Longitudinal reinforcement and transverse reinforcement[3]	$\frac{\text{Shear}}{t_w l_w \sqrt{f_c'}}$									
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	0.018	0.030	0.60	0.006	0.009	0.012	0.015	0.015	0.024
	≥ 6	0.012	0.020	0.30	0.004	0.006	0.008	0.010	0.010	0.016
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	0.012	0.025	0.40	0.006	0.007	0.008	0.010	0.010	0.020
	≥ 6	0.008	0.014	0.20	0.004	0.005	0.006	0.007	0.007	0.012

1. For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 6-4 and 6-5.
2. For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be $\leq 0.15 A_g f_c'$; otherwise, the member must be treated as a force-controlled component.
3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the beam at a spacing $\leq d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of beam.

Table 7-5: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Members Controlled by Shear

failure to collapse. All connectors for load-bearing and non-load-bearing walls will therefore be designed in accordance with ACI 318, as modified by Section 9.1.1 of FEMA 302. The shear force will be uniformly distributed throughout the height or length of the shear wall with reasonably spaced connectors (maximum spacing 4 feet), rather than with a few that will have localized concentration of stresses. Detailed calculations will be made, including the localized effects in concrete walls attributed from these connectors. Sufficient details of connectors and embedded anchorage will be provided to preclude construction deficiency.

(4) Typical details. Refer to Figure 7-9 for details.

(5) Acceptance criteria. FEMA 302 requires that connections for precast concrete walls shall be designed to be stronger than the adjacent precast panels. The lateral-load response behavior is therefore comparable to that for monolithic shear walls, and the acceptance criteria of Paragraph 7-2f(3) will be applicable.

h. Masonry Shear Walls.

(1) General design criteria. This section prescribes the criteria for the structural design of shear walls of unit masonry construction. The basic reference documents are FEMA 302 and ACI 530.

(2) Unreinforced or plain masonry bearing walls or shear walls, where permitted, shall be used only for buildings in Seismic Design Category A or B. Design shall be in accordance with Section 11.3.3 of FEMA 302.

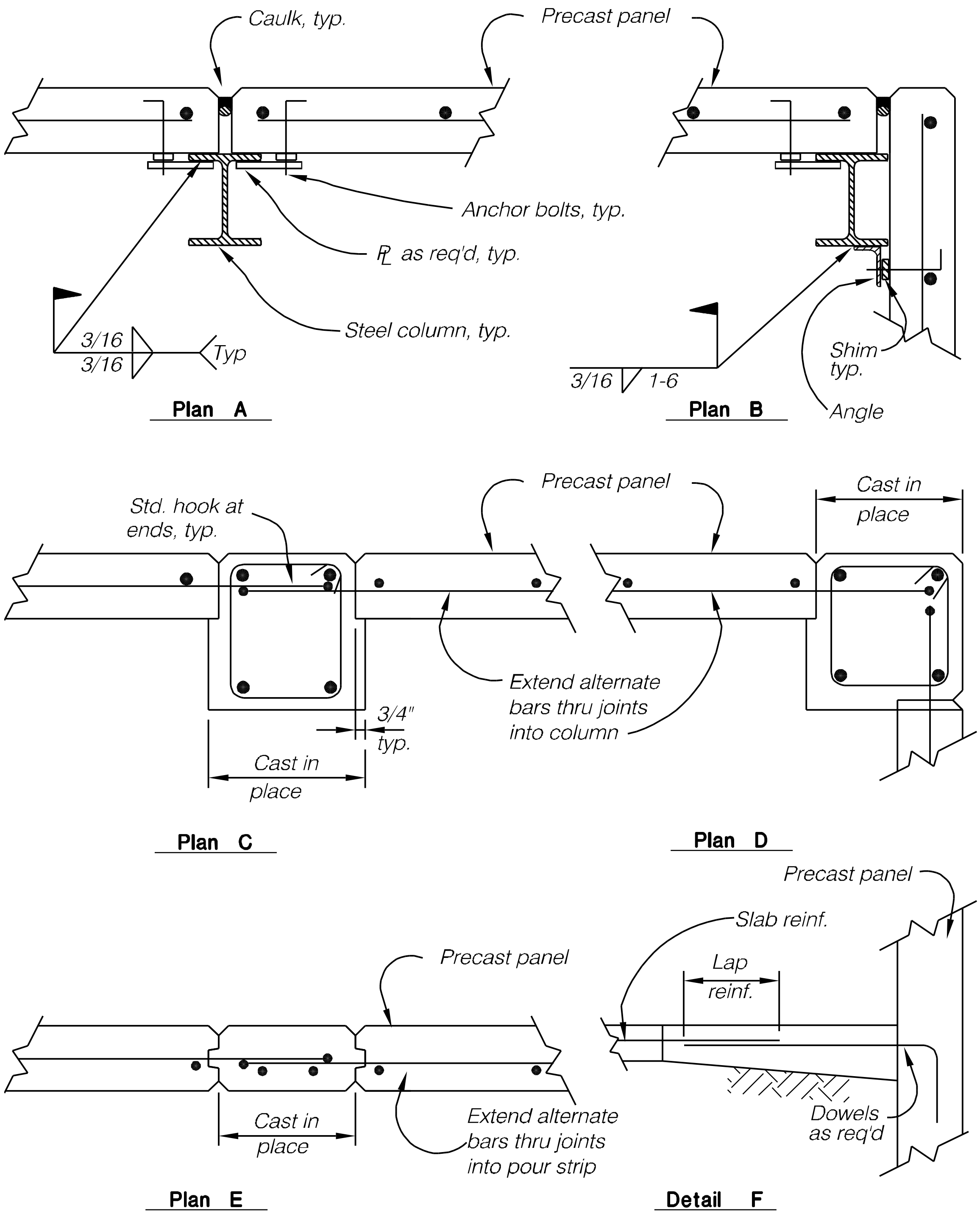
(3) Masonry construction prescribed by this document shall be in accordance with the following provisions:

(a) Seismic Design Categories A, B, and C. Masonry shear walls shall comply with the requirements of intermediate reinforced masonry shear walls (Section 11.11.4 of FEMA 302) or special reinforced masonry shear walls (Section 11.11.5 of FEMA 302).

(b) Seismic Design Categories D, E, and F. Masonry shear walls shall comply with the requirements of special reinforced masonry shear walls (Section 11.11.5 of FEMA 302).

(c) Basic requirements. Unit masonry will be reinforced with deformed bars for axial, flexural, shear, and diagonal tension stresses as determined by design calculations. Additional reinforcing bars are prescribed for use around openings, at corners, at anchored intersections, and at the ends of wall panels (for example, at control joints). The minimum reinforcement prescribed in FEMA 302 is intended to provide empirical requirements relative to damage control (ductility and boundary conditions). Layout and details of construction will be compatible with the application of the rules for modular measure.

(d) Types of reinforced masonry walls. Masonry will conform to one of the following basic types: reinforced grouted masonry, reinforced hollow masonry, or reinforced filled-cell masonry.



1 inch = 25mm

Figure 7-9 Tilt-up and other precast concrete walls - typical details of attachments

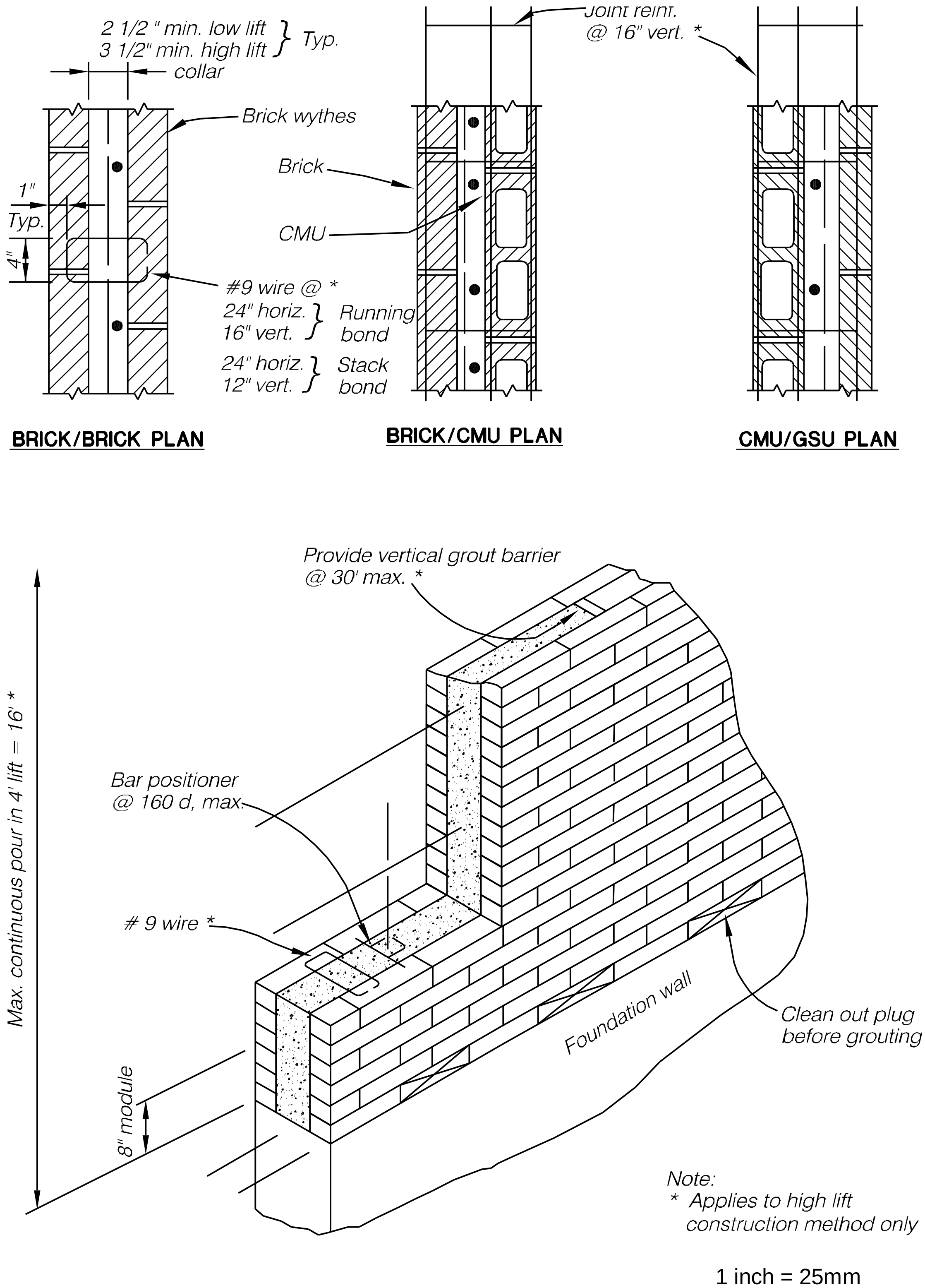
1. Reinforced grouted masonry is that type of construction made with two wythes of masonry units in which the collar joint between is reinforced and filled solidly with concrete grout. The grout may be placed as the work progresses or after the masonry units are laid. Collar joints will be reinforced with deformed bars, both vertical and horizontal. Reinforcement and embedded items such as structural connections and electrical conduit shall be positioned so as to allow proper placement of grout. All units will be laid in running bond with full shoved head and bed mortar joints. Masonry headers will not project into grout spaces. Clipped-brick headers will be used where the appearance of masonry headers is required (see Figure 7-10).

2. Reinforced hollow masonry is that type of construction made with a single wythe of hollow masonry units (concrete or clay blocks), reinforced vertically and horizontally with steel bars, and cores and voids containing reinforcing bars or embedded items are filled with grout as the work progresses (see Figure 7-11).

3. Reinforced filled-cell masonry is that type of construction made with a single wythe of hollow masonry units, reinforced vertically and horizontally with deformed steel bars, and all cores and voids are filled solidly with grout after the wall is laid (see Figure 7-12).

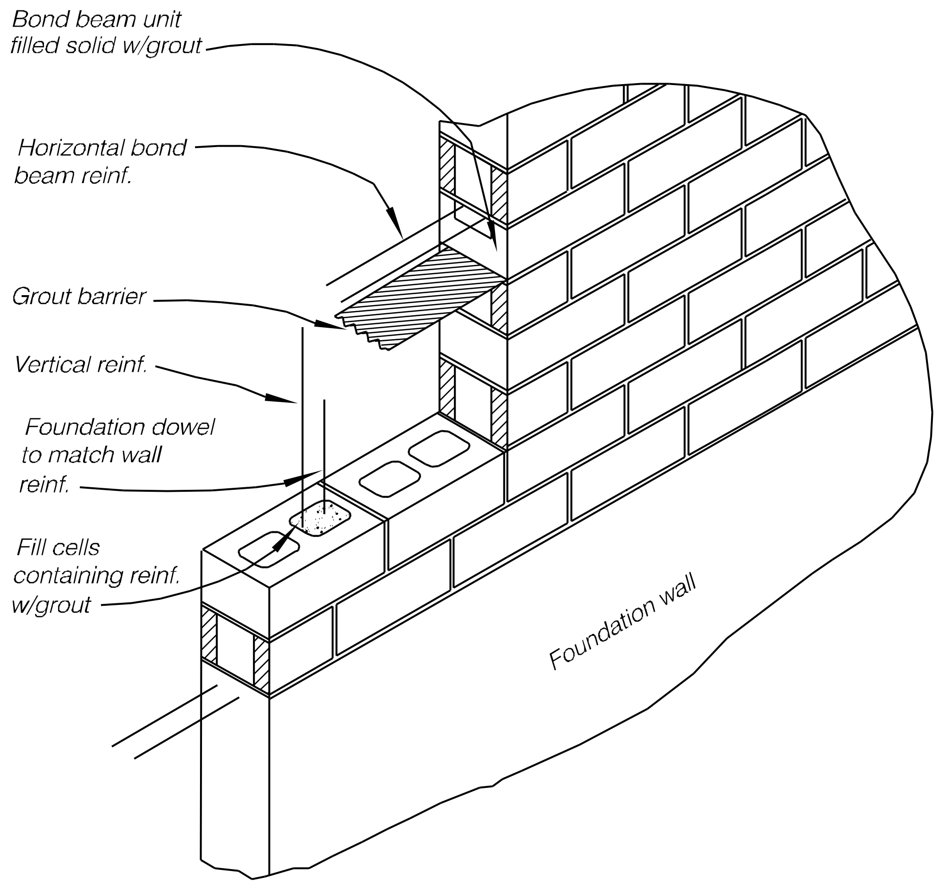
(e) Bond beams. Bond beams will be located as indicated in Figure 7-13. Reinforcement bars in bond beams will be lapped as prescribed in ACI 530 at splices, at intersections, and at corners. Bar splices will be staggered. Bond beams will be provided at top of masonry foundation wall stems, below and at top of openings or immediately above lintels, at floor and roof levels, and at top of parapet walls. Intermediate bond beams will be provided as required to conform to the maximum spacing of horizontal bars. When, however, the height is not a multiple of this normal spacing, the spacing may be increased up to a maximum of 24 inches (610mm) , provided the bond beams are supplemented with joint reinforcement. One line of joint reinforcement will be provided for each 8-inch (203mm) increase in the spacing. No additional bond beam will be required between window openings that do not exceed 6 feet (1.8m) in height, provided the prescribed supplemental joint reinforcement is installed. To facilitate the placement of steel or concrete core fill, the top bond beam for filler walls or partitions may be placed in the next-to-top course. The area of bond beam reinforcement will be included as part of the minimum horizontal steel.

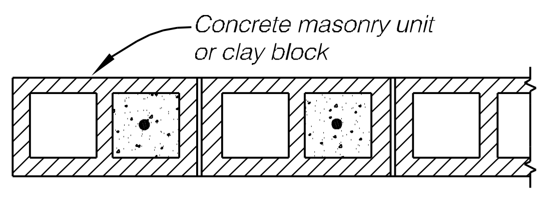
(f) Design for crack control. Guidelines provided in TM 5-809-3/NAVFAC DM-2.9/AFM 88-3, Chapter 3, will be utilized to minimize cracking of masonry walls due to drying shrinkage and thermal expansion and contraction. The placement of control joints must be coordinated with the seismic design. Because the control joints provide a complete separation of the masonry, the location of control joints fixes the length of wall panels, and in turn, the rigidity of the walls, the distribution of seismic forces, and the resulting unit stresses. Therefore, adding, eliminating, or relocating control joints will not be permitted once the structural design is complete. Control joints will never be assumed to



1 inch = 25mm

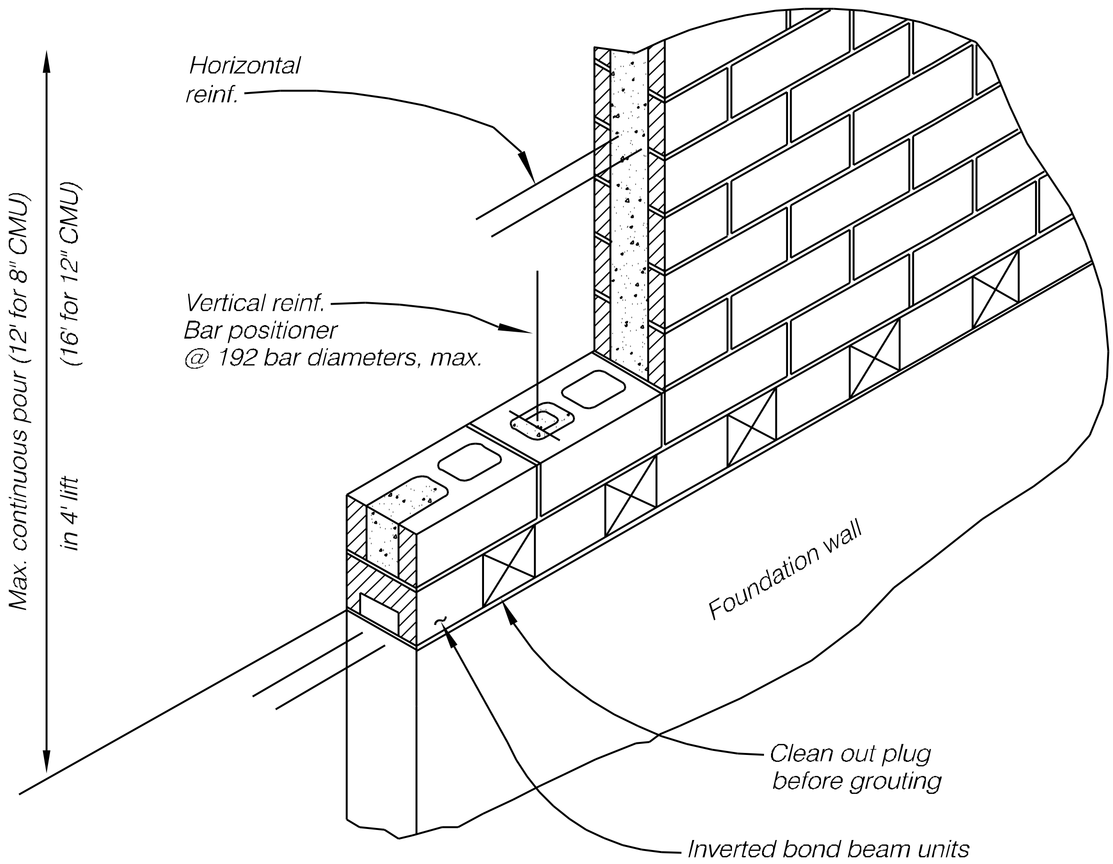
Figure 7-10 Reinforced grouted masonry.



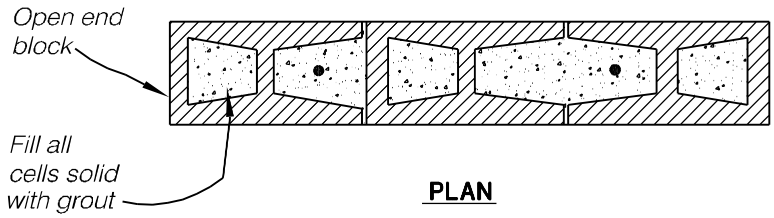


PLAN

Figure 7-11 Reinforced hollow masonry.



REINFORCED FILLED CELL MASONRY
NOTE: DETAILS SHOWN ARE HIGH LIFTS.



PLAN

1 inch = 25mm
1 foot = 0.30m

Figure 7-12 Reinforced filled-cell masonry.

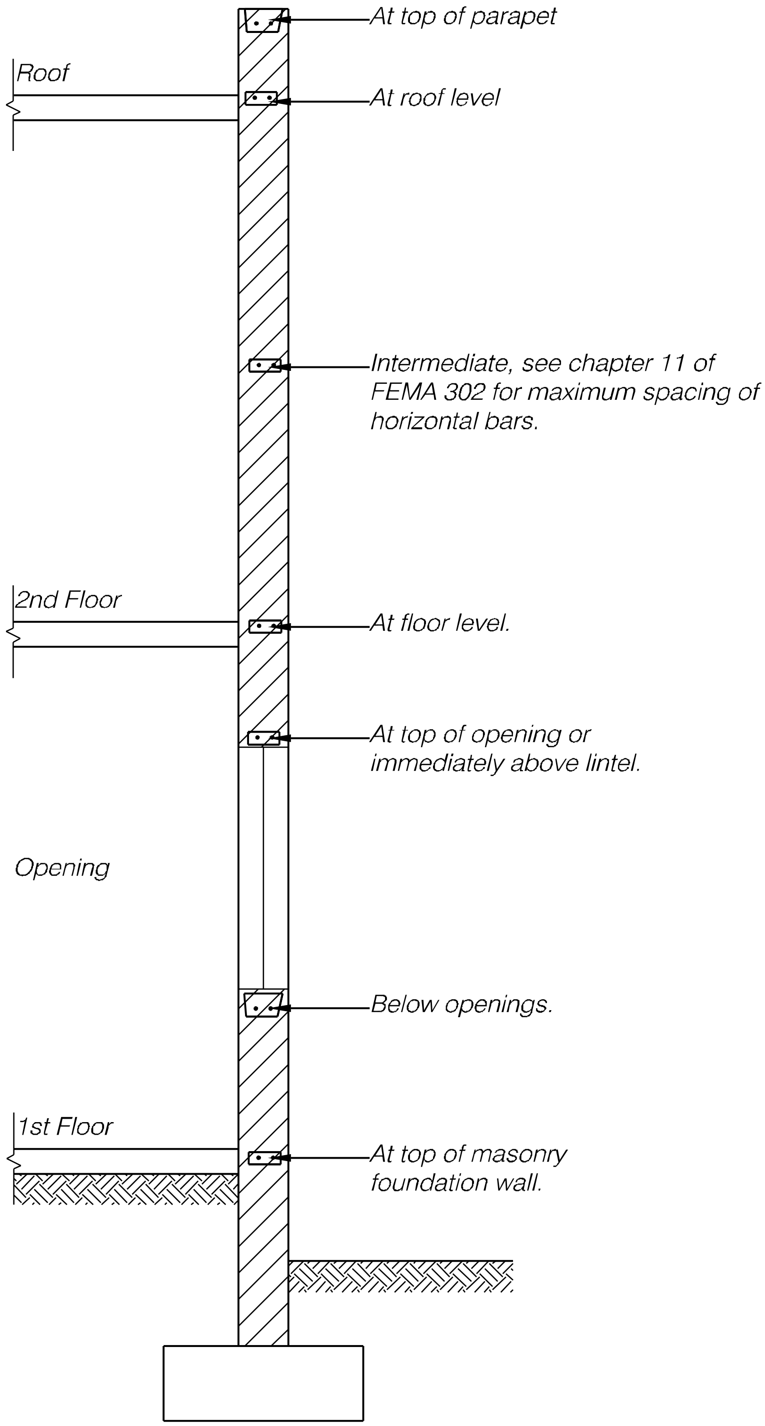


Figure7-13 Location of Bond Beams.

transfer bending moments or diagonal tension across the joint: joint reinforcement and bars in nonstructural bond beams will be terminated at control joints. Deformed bars in structural bond beams (those acting as chords and collectors) will be made continuous for the length of the diaphragm (refer to Figure 7-14).

(g) Design considerations.

1. Wall weights. Refer to ACI 530 for the average weight of concrete masonry units and the average weight of completed walls.

2. Shearing stresses in hollow masonry shall be based on area of the grouted cores plus the minimum net bedded cross-sectional area of the members under consideration.

3. Boundary Zones. When the compressive strains in special reinforced concrete shear walls exceed 0.0015 under combined loads, boundary zones shall be provided as prescribed for special reinforced concrete shear walls in Paragraph 7-2f(3).

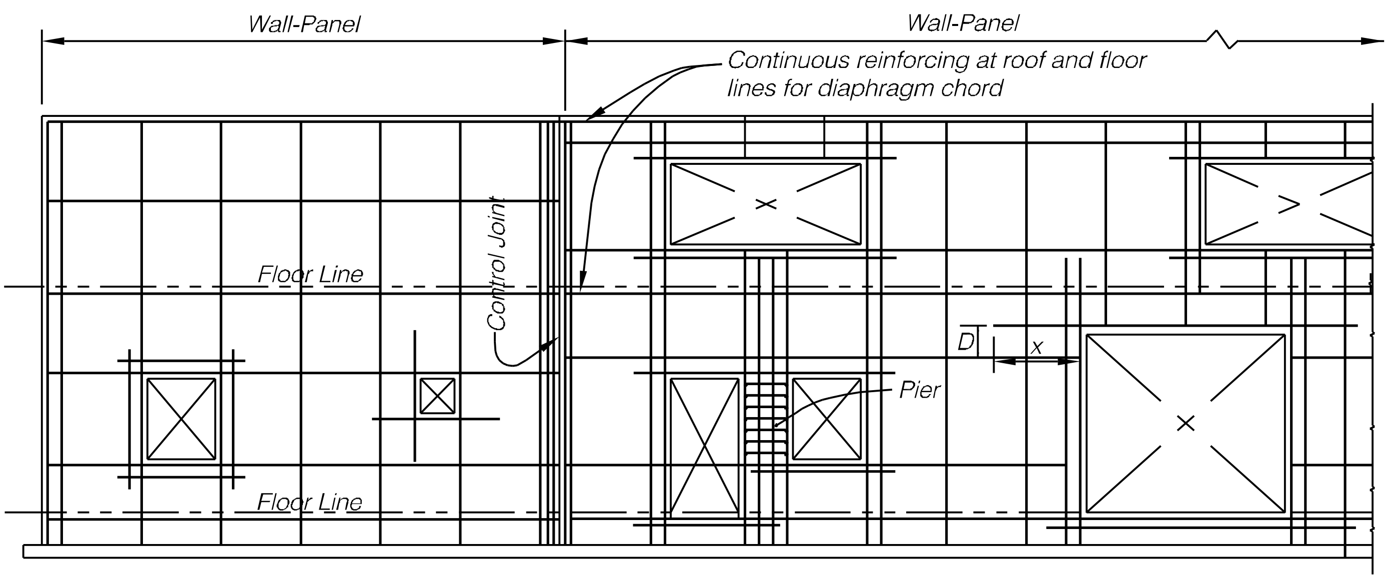
(h) Reinforcing. Typical reinforcement is shown in Figure 7-16.

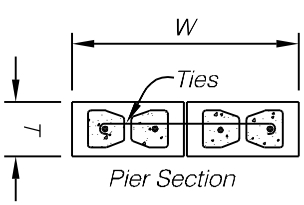
1. Minimum reinforcing. Unit masonry must be reinforced not only for structural strength, but to provide ductile properties and to hold it together in the event of severe seismic disturbance. All walls and partitions will be reinforced as required by structural calculations, but in no case with less than the minimum area of steel and the maximum spacing of bars prescribed in Chapter 11 of FEMA 302. Only reinforcement that is continuous in any wall panel will be considered in computing the minimum area of reinforcement. Joint reinforcement used for crack control or mechanical bonding may be considered as part of the total minimum horizontal reinforcement, but will not be used to resist computed stresses. Further additional bars will be provided around openings, at corners, at anchored intersections in wall piers, and at ends of wall panels, as prescribed elsewhere in this chapter. Vertical bars in walls will be spliced as prescribed in ACI 530.

2. Reinforcing in shear walls. In special reinforced masonry shear walls, reinforcement required to resist in-plane shear will be terminated with a standard hook or with an extension of proper embedment length beyond the reinforcing at the end of the wall section. The hook or extension may be turned up, down, or horizontally. Provisions will be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams will be fully anchored into these elements.

3. Reinforcing in wall piers. Horizontal reinforcement will be in the form of ties as shown in Figure 7-16.

4. Column ties. For buildings in Seismic Design Categories D, E, and F, the spacing of column ties will not be more than 8 inches (203mm) for the full height for columns stressed by tensile or compressive axial overturning forces due to the seismic loads of Chapter 3; and 8 inches (203mm) for the tops and bottoms of all other columns for a distance of one-sixth of the clear column height, but not less than 18 inches (457mm), nor





Elevation of a typical wall

x = 2D + splice length
but not less than 24" (610mm)

Figure 7-14: Typical wall reinforcement - reinforced masonry.

the maximum column dimension. Tie spacing for the remaining column height will be not more than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches (457mm). Hooks in column ties will have a minimum turn of 135 degrees plus an extension of at least six bar diameters, but not less than 4 inches (102mm) at the free end of the bar, except that where the ties are placed in the horizontal bed joints, the hook will consist of a 90-degree bend having a radius of not less than four bar diameters, plus an extension of 32 bar diameters.

5. Reinforcing in stacked bond. For buildings in Seismic Design Categories A, B, and C, the minimum horizontal reinforcement ratio shall be .0007 bt. This ratio shall be satisfied by uniformly distributed joint reinforcement fully embedded in mortar or by horizontal reinforcement spaced not over 4 feet (1.2m), and fully embedded in grout. For buildings in Seismic Design Categories D, E, and F, the minimum horizontal reinforcement ratio shall be 0.015 bt. If open end units are used and grouted solid, then the minimum horizontal reinforcement ratio shall be .0007 bt. The above reinforcement ratios may be satisfied by combinations of joint and horizontal reinforcement.

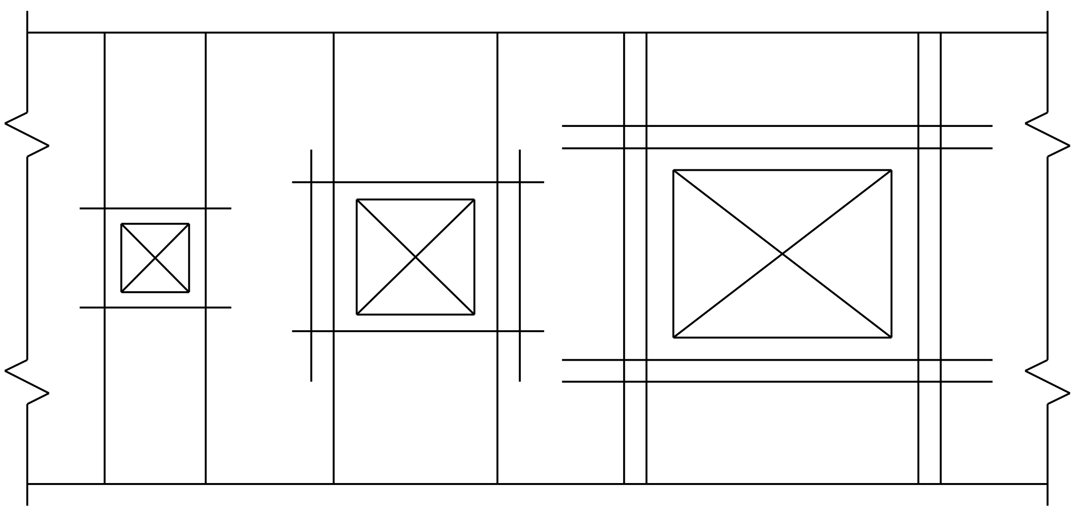
6. Reinforcing at wall openings. Since the area around wall openings is vulnerable to failure, supplemental reinforcement is prescribed herein. For purposes of this paragraph, the term "jamb bars" will mean bars of the same size, number, extent, and anchorages as the typical vertical stud reinforcement in that wall, and in no case less than one bar, #4 (10M) or larger (refer to Figure 7-15).

i. Case I. Case I applies to all openings in nonstructural partitions over 100 square inches (64.5×10^3 mm^2), and any opening in structural partitions or exterior walls that is 2 feet (0.6m) or less both ways, but over 100 square inches. Jamb bars will be provided on each side of the opening, and at least one bar, #4 (10M) or larger, will be provided at top and bottom of the opening. The lintel bars above the opening may serve as the top horizontal bar, and a bond beam bar at the bottom of the opening may serve as the bottom horizontal bar.

ii. Case II. Case II applies to exterior walls and structural partitions for any opening that exceeds 2 feet (0.6m), but is not over 4 feet (1.2m) in any direction. The perimeter reinforcement will be the same as in Case I, plus additional reinforcement as follows: #4 (10M) or larger will be provided on all four sides of the opening, in addition to the bars required in Case I, and shall extend not less than 40 bar diameters or 24 inches (0.6m), whichever is larger, beyond the corners of the opening.

iii. Case III. Case III applies to any opening that exceeds 4 feet (1.2m) in either direction in exterior walls or structural partitions. The perimeter reinforcement will be the same as in Case II, except that vertical jamb bars will be provided in lieu of the shorter vertical bars.

(i) Additional details (see Figure 7-16).



Case I Case II Case III

Refer to paragraph 7-2a(3)f for application of Cases I, II and III.

Figure 7-15 Reinforcement around wall openings.

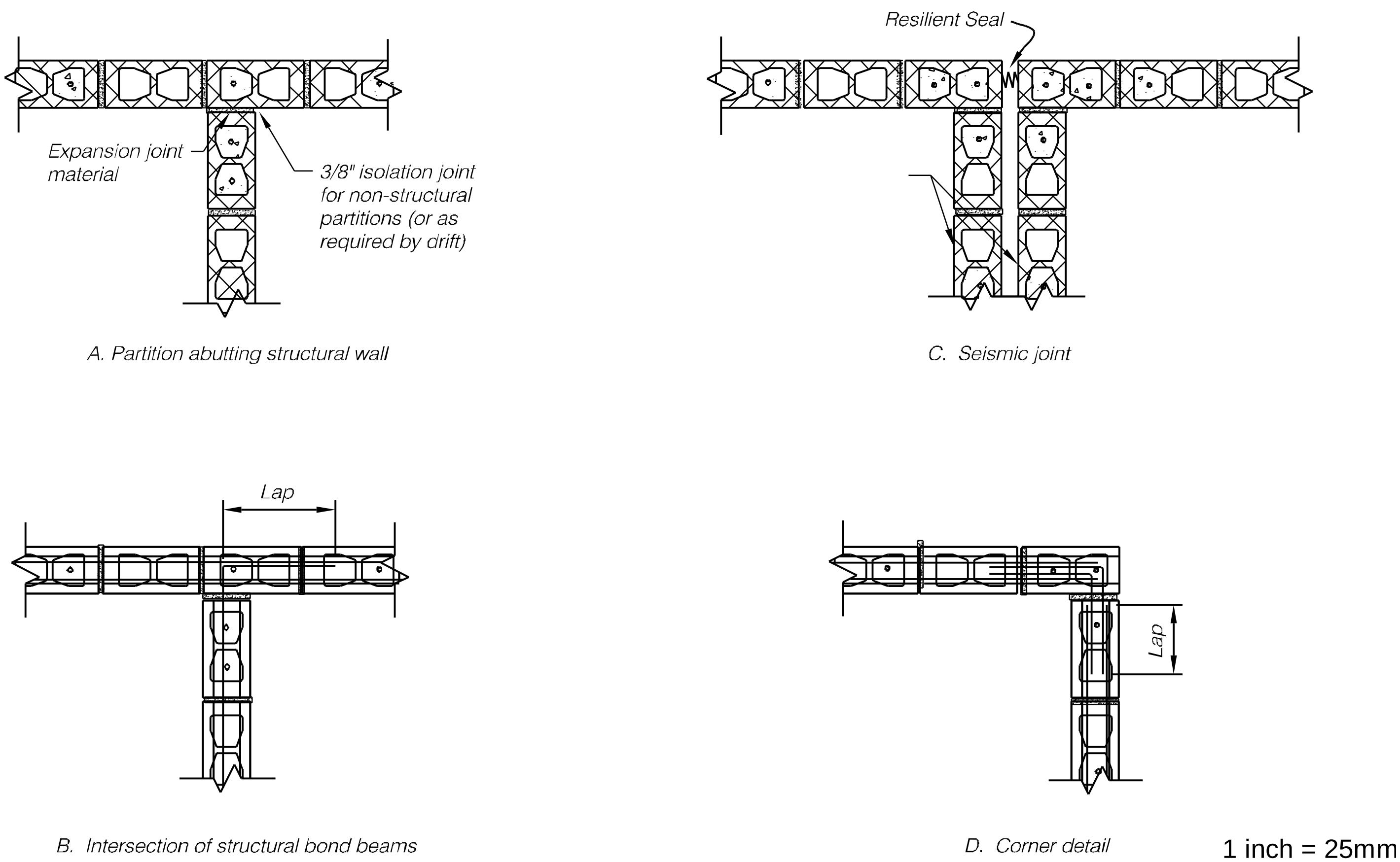
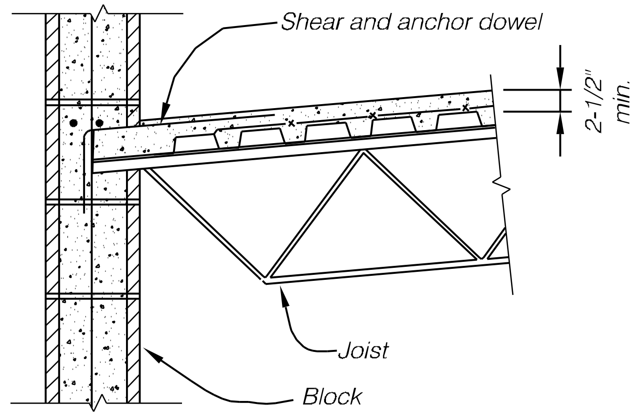
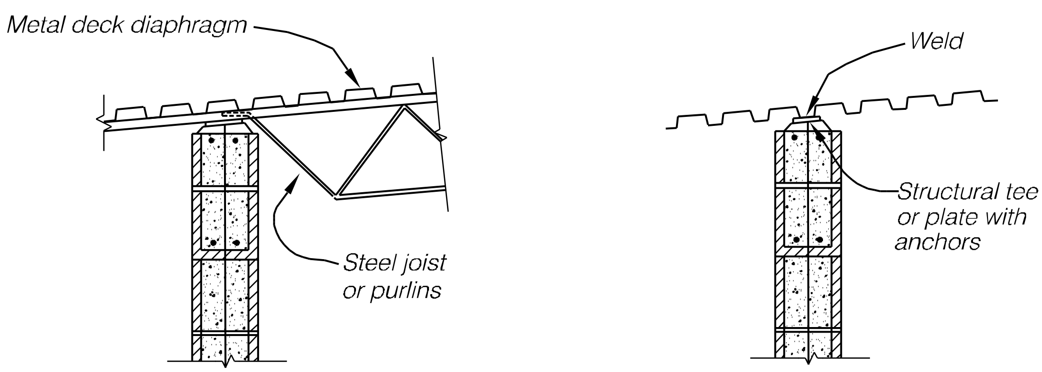


Figure 7-16 Masonry wall details.

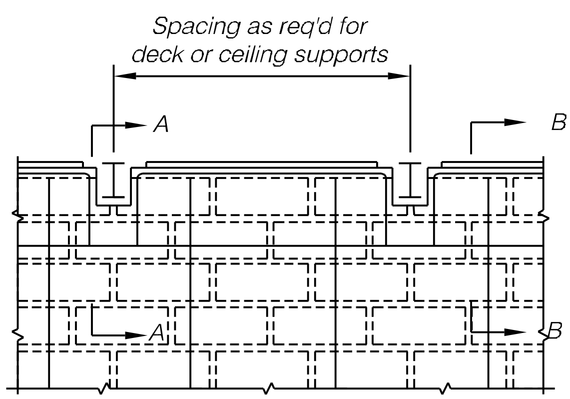


CONCRETE ON METAL FORM



SECTION A-A

SECTION B-B



ELEVATION

MASONRY WALL TO METAL DIAPHRAGM

1 inch = 25mm

Figure 7-16 continued.

(4) Special requirements.

(a) Excluded materials. The following materials will not be used as part of the structural system:

1. In areas where $S_{DS}\ \$0.50$, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than $1^{1}/_{4}$ parts by volume of hydrated lime or lime putty per one part of Portland cement.

2. In areas where $S_{DS}\ \$\ 0.75$, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than 1/2 part by volume of hydrated lime or lime putty per one part of Portland cement.

(b) Stacked bond. Since a running bond pattern is the strongest and most economical, the criteria in this document are based upon each wythe of masonry being constructed in a running bond pattern. The use of a stacked bond pattern will be restricted to reinforced walls essential to the architectural treatment. Filled-cell masonry or grouted masonry will be used. For filled-cell masonry, open-end blocks will be used and so arranged that closed ends are not abutting, and all head joints are made solid, and bond beam units shall be used to facilitate the flow of grout.

(c) Height limit. Unit masonry construction designed in accordance with the empirical procedure of ACI 530 will not be used for shear walls where the height of the building exceeds the limits given in Table 7-6.

(d) Joint reinforcement. Joint reinforcement will not be used in the calculation of shear strength.

(e) Mechanical splices. Mechanical splices will develop 125 percent of the specified yield strength of the bar in tension, except that for compression bars in columns that are not part of the seismic system and are not subject to flexure, the compressive strength only need be developed.

(f) Cavity walls. Cavity walls are not practical for use as shear walls because each wythe individually, and both wythes acting together in proportion to their relative rigidities, must be capable of carrying the required loads. It is usually much more economical to construct a two-wythe cavity-type wall by using an interior structural wythe and an exterior nonstructural anchored veneer wythe. See Chapter 10 for requirements for anchored veneer.

(g) Drawings. The locations of control joints and the identification of structural and nonstructural walls and partitions for all masonry construction will be shown on preliminary and contract drawings. On contract drawings, complete details for masonry, reinforcement, and connections to other elements will be shown. Detailing procedures outlined in ACI 318 are generally applicable to reinforced masonry.

(5) Acceptance criteria.

(a) Response modification factors, *R*, for Performance Objective 1A are provided in Table 7-1.

Construction	Maximum l/t or h/t
Bearing Walls	
Solid units or Fully grouted	20
All other	18
Nonbearing walls	
Exterior	18
Interior	36

Table 7-6. Lateral Support Requirements for Masonry Walls.