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BSSC 82-2

AMENDMENTS TO ATC 3-06 TENTATIVE PROVISIONS FOR THE DEVELOPMENT OF SEISMIC REGULATIONS FOR BUILDINGS FOR USE IN TRIAL DESIGNS

This report documents the deliberations of a group of professionals jointly selected by the Building Seismic Safety Council and the National Bureau of Standards and is for use in modifying Tentative Provisions for trial designs.

DECEMBER 1982

Any opinions, findings, conclusions, or recommendations expressed in this publication do not necessarily reflect the view of the Federal Emergency Management Agency.

NATIONAL BUREAU OF STANDARDS

BSSC
BUILDING SEISMIC SAFETY COUNCIL

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Prepared for use by the:

BUILDING SEISMIC SAFETY COUNCIL

Sponsored by:

FEDERAL EMERGENCY MANAGEMENT AGENCY

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National Bureau of Standards
Washington, DC 20234

Building Seismic Safety Council
Washington, DC 20005

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NATIONAL BUREAU OF STANDARDS, Ernest Ambler, *Director*

PREFACE AND ACKNOWLEDGMENTS

This report was developed by the BSSC/NBS Committee 12 - Trial Design Overview Committee, formed jointly by the Building Seismic Safety Council and the National Bureau of Standards. The membership of the committee consisted of:

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Other individuals on BSSC Technical Committees, too numerous to acknowledge by name, offered suggestions and improved the report. Their contributions are gratefully appreciated.

TABLE OF CONTENTS

INTRODUCTION

The report Tentative Provisions for the Development of Seismic Regulations for Buildings[1] (Tentative Provisions) was developed by the Applied Technology Council (ATC) in a project funded by the National Science Foundation and the National Bureau of Standards (NBS). The Tentative Provisions include many innovations, and this departure from present building design practice dictates a need for careful assessment. The authors of the Tentative Provisions realized this, stating in their preface:

> "Because of the many new concepts and procedures included in these Tentative Provisions, they should not be considered for code adoption until their workability, practicability, enforceability, and impact on cost are evaluated by producing and comparing building designs for the various design categories included in this document."

Interest in provisions for the seismic safety of buildings has grown rapidly in recent years, and this led to the establishment of the Building Seismic Safety Council (BSSC) in 1979. BSSC has founded under the auspices of the National Institute of Building Sciences to provide a national forum to foster improved seismic safety provisions for buildings.

The BSSC and the NBS conducted a review and refinement project to improve the Tentative Provisions where possible before beginning the assessment process called for by the authors of the report. The review was conducted using a committee structure with interests that collectively covered the Tentative Provisions. Recommendations[2] of these committees were presented to the BSSC at an annual meeting in November of 1979 and subsequently balloted by the BSSC.

The BSSC assigned the task of resolving ballot issues and recommending a set of provisions for use in trial designs to a newly formed Trial Design Overview Committee sponsored jointly by the BSSC and NBS. The Trial Design Overview Committee was also assigned the task of completing a plan[3] for a trial design program for the purpose of evaluating the amended Tentative Provisions.

In order to recommend a set of provisions and conduct the trial designs in a timely fashion, the Trial Design Overview Committee took the position that it was not feasible to completely resolve all ballot issues prior to the conduct of the trial design program. In some instances, it was considered necessary to have the results of the program in order to resolve the issues. Thus, the Overview Committee took the approach of accepting the results of the BSSC ballots in order to determine what changes would be made for the purpose of conducting trial designs. It was also considered essential to maintain a consistent set of provisions. In the case of Chapters 1 through 11, 13, and 14 this was accomplished simply by accepting the ballot results. No internal conflicts were created by doing this. However, in the case of the recommendations for masonry, Chapter 12 and 12A, this was not the case. Accordingly, a task group of the Trial Design Overview Committee was established to review the ballot results for those two chapters and recommend a set of consistent provisions.

A task group was formed which invited the BSSC Technical Committee on Masonry to attend its deliberations and provide additional input. The approach taken by the task group was to accept ballot items which passed the BSSC ballot process and consider the items which did not pass and modify them to insure consistency. In general, this meant that items were reverted back to the original ATC wording although location of the material within the chapters might be somewhat different. The final set of recommendations for Chapters 12 and 12A are considered internally consistent and in keeping with the spirit of the BSSC ballot process.

This report is prepared in three parts. Part A consists of a completely revised Chapter 12 - Masonry which is intended to replace entirely Chapter 12 in the ATC report. The text is preceded by a list of the sections and the corresponding BSSC ballot number. The original ballot number and item may be found in reference 2. This procedure was selected in order to insure traceability

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[1] Applied Technology Council, Tentative Provisions for the Development of Seismic Regulations for Buildings, National Bureau of Standards, Special Publication 510, Washington, DC, 1978.

[2] Leyendecker, E. V., and Harris, J. R., editors, "Review and Refinement of ATC 3-06 Tentative Seismic Provisions: Report of the Joint Committee on Review and Refinement," National Bureau Standards, NBSIR 80-2111-11, Washington, DC, 1980.

[3] Harris, J. R. and Leyendecker, E. V., editors, "Plan for a Trial Design Program to Assess Amended ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings," NBSIR 82-2589 and BSSC 82-1, November 1982, Washington, DC.

through the entire process of evaluation. Part B contains the amendments to Chapter 12A - Reference Guideline for Masonry Construction. Once again, this text completely replaces Chapter 12A in the the ATC report. As in the case of Part A, the various sections and the BSSC ballot number are identified in the listing preceding the text. Part C contains the amendments to Chapters 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 13, and 14. These amendments have been printed on one side only and can be readily cut and pasted into the original ATC report. As in the case of Parts A and B, a listing of changes is presented with the BSSC ballot number.

It is reiterated that amended the <u>Tentative Provisions</u> are being evaluated in the trial design program. This evaluation is being done in order to consider their "workability, practicability, enforceability, and impact on cost." These results will be considered by the BSSC as it prepares a resource document of seismic design provisions.

PART A - AMENDMENTS TO CHAPTER 12

Listing of Amendments to Chapter 12

Section with Modification[1]		BSSC Ballot Number
	BACKGROUND	5/1
12.1	REFERENCE DOCUMENTS	NC[2]
12.2	STRENGTH OF MEMBERS AND CONNECTIONS	5/2-C[3]
12.2.1	UNREINFORCED MASONRY DESIGN	5/3-C
12.2.2	REINFORCED MASONRY DESIGN	5/3-C
12.3	SEISMIC PERFORMANCE CATEGORY A	5/4
12.4	SEISMIC PERFORMANCE CATEGORY B	5/5
12.4.1	CONSTRUCTION LIMITATIONS	
	(A) DESIGN	5/5
	(B) TIES	5/6
	(C) SHEAR WALLS	5/6
	(D) SCREEN WALLS	5/6
	(E) NONSTRUCTURAL COMPONENTS	5/7
	(F) CONSTRUCTION TYPE	5/7-C
	(G) GROUTED MASONRY - MULTIWYTHE	5/10
	(H) REQUIRED STRENGTH FOR MORTAR & GROUT	5/12-ATC[4]
	(I) JOINTS	5/12
	(J) GLASS MASONRY	5/13
	(K) REINFORCEMENT DEVELOPMENT, ANCHORAGE AND SPLICES	5/14
	(L) DISTRIBUTION OF CONCENTRATED LOADS	5/15
12.4.2	MATERIAL LIMITATIONS	5/15
12.5	SEISMIC PERFORMANCE CATEGORY C	5/16
12.5.1	CONSTRUCTION LABORATORY	
	(A) REINFORCEMENT	5/16
	(B) TIE ANCHORAGES	5/16

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[1] Normally the major section such as 12.6 is listed. Subsections such as 12.4.1 are listed only if necessary to adequately identify ballot items.

[2] NC indicates no change proposed from original ATC.

[3] Indicates the ballot item passed but was modified slightly for internal consistency.

[4] Used original ATC wording to resolve an item that did not pass the ballot.

Section with Modification		BSSC Ballot Number
	(C) REINFORCED COLUMNS	5/16
	(D) SHEAR WALL BOUNDARY ELEMENT	5/16-C
	(E) JOINT REINFORCEMENT	5/16
	(F) STACK BOND CONSTRUCTION	5/16
	(G) WALLS	5/17
	(H) HOLLOW UNIT MASONRY	5/17
	(I) SHRINKAGE OF CONCRETE UNITS	5/19-ATC
	(J) CORE TESTS FOR SHEAR BOND IN GROUTED MASONRY - MULTIWYTHE	5/21
	(K) MASONRY WALLS	5/24
12.5.2	MATERIALS LIMITATIONS	5/22-ATC, 5/23, 5/25
12.6	SEISMIC PERFORMANCE CATEGORY D	5/26
12.7	SHEAR WALL REQUIREMENTS	5/27
Table 12.1		5/28-ATC

The following ballot items were deleted due to redundancy with Chapter 12A: 5/8, 5/9, 5/11, 5/18, and 5/20. Portions of ballot items 5/10, 5/21, and 5/26 were deleted as being redundant with Chapter 12A.

CHAPTER 12

MASONRY

BACKGROUND

The masonry design and construction procedures given in this Chapter and Chapter 12A are essential to provide seismic performance levels for the seismic forces exclusively for the purpose of this document.

12.1 REFERENCE DOCUMENTS

The quality and testing of masonry and steel materials and the design and construction of masonry and reinforced masonry components which resist seismic forces shall conform to the requirements of Chapter 12A and the references listed therein except as modified by the provisions of this Chapter. For definitions, see Sec. 12A.1.1.

12.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, ϕ, and 2.5 times the allowable working stresses of Chapter 12A or as these allowable are further modified by this chapter. The value of ϕ shall be as follows:

When considering axial or flexural compression and bearing stresses in the masonry.	$\phi = 1.0$
For reinforcement stresses except when considering shear.	$\phi = 0.8$
When considering shear carried by shear reinforcement and bolts.	$\phi = 0.6$
When considering masonry tension parallel to the bed joints, i.e., horizontally in normal construction.	$\phi = 0.6$
When considering shear carried by the masonry.	$\phi = 0.4$
When considering masonry tension perpendicular to the bed joints, i.e., vertically in normal construction.	$\phi =$ Zero

Stresses entitled "special inspection" in Chapter 12A shall only be used when the work is fully inspected per Sec. 1.6.2, 1.6.4, and 12A.7.

12.2.1 UNREINFORCED MASONRY DESIGN

Unreinforced masonry designed in accordance with Sec. 12A.6.1 shall be assumed to be cracked in the tension zone. The resultant linear distribution of compressive stresses must be in equilibrium with the applied forces and the maximum compressive stress must not exceed the values of Table 12A-2.

EXCEPTION:
Bed joints of unreinforced vertical components constructed using stacked bond, which are subjected to bending in the plane of the component, shall remain uncracked.

12.2.2 REINFORCED MASONRY DESIGN

Reinforced masonry shall be designed and constructed in accordance with one of the following three procedures and the provisions of other Sections of this Chapter.

(A) MASONRY DESIGNED AND REINFORCED AS REQUIRED. Specific reinforced masonry areas or elements may be considered as resisting stresses in accordance with the design criteria in Section 12A.6.3 and allowable stresses in Table 12A-4 for reinforced masonry provided such elements fully comply with the design and construction requirements for reinforced masonry. When these wall areas or elements contain less reinforcement than required in Section 12.2.2(C), allowable shear stresses for unreinforced masonry shall be used.

12.2.2 Cont.

The width of these elements, tributary to the reinforcement, must meet the requirement of effective width of masonry given in Section 12A.6.3(A). The R factor of Table 3-B shall be as required for unreinforced masonry unless all masonry structural elements are reinforced in accordance with Section 12.2.2(C).

(B) MASONRY DESIGNED AND REINFORCED AS REQUIRED AND CONTAINING NOMINAL PRESCRIBED REINFORCING. This masonry shall conform to all of the requirements of Section 12.2.2(A) and to the additional requirements of this Section. Construction shall be grouted masonry -- multiwythe or hollow unit masonry containing reinforcement as specified below. Masonry joint reinforcement shall be, and ties may be, embedded in the mortar in the bed joints. All other reinforcement shall be embedded in grout. Minimum masonry, mortar, and grout coverages applicable to reinforced masonry shall be provided. Only Type M or S mortar shall used. The R factor of Table 3-B shall be as required for unreinforced masonry unless all masonry structural elements are reinforced in accordance wih Section 12.2.2(C).

Reinforcing for columns shall conform to the requirement of Sec. 12A.6.3(F). For walls the maximum spacing of vertical reinforcement shall be 8 feet where the nominal thickness is 8 inches or greater and 6 feet where the nominal thickness is less than 8 inches. Vertical reinforcement shall also be provided each side of each opening and at each corner of all walls. Horizontal reinforcement not less than 0.2 square inch in area shall be provided at top of footings, at the bottom and top of wall openings, near roof and floor levels, and at the top of parapet walls and, where distributed joint reinforcement is not provided, at a maximum spacing of 12 feet where the nominal masonry thickness is 8 inches or greater and 9 feet where the nominal thickness is less than 8 inches. The vertical reinforcement ratio and the horizontal reinforcement ratio shall each be not less than 0.00025. Where not prohibited by Chapter 12A or this Chapter, stacked bond construction may be used. When stacked bond is used the minimum horizontal reinforcement ratio shall be increased to 0.0007. This ratio shall be satisfied by masonry joint reinforcement spaced not over 16 inches or by reinforcement embedded in grout spaced not over 4 feet. Reinforcement shall be continuous at wall corners and intersections.

Splices for reinforcement shall conform to all requirements for splices in reinforced masonry.

(C) MASONRY DESIGNED AND REINFORCED WITH PRESCRIBED MINIMUM AREAS IN ADDITION TO THE REQUIREMENTS OF 12.2.2(B). This additional reinforcement shall be in both horizontal and vertical directions. The sum of the areas of reinforcement in both directions shall be at least equal to 0.002 times the gross cross-section of the masonry with at least 0.0007 times the gross cross-sectional area of the masonry in each direction.

Where reinforcement ratios are less than those in the in the preceding paragraph the allowable stresses for unreinforced masonry shall be used. If reinforcement ratios are equal to or greater than these ratios, the stresses of Table 12A-4 may be used and the R factors in Table 3-B for reinforced masonry may be used.

The reinforcement shall be limited to a maximum spacing of 4 feet on center. The minimum diameter of reinforcement shall be 3/8 inch except that joint reinforcement may be considered as part of the required minimum reinforcement.

Horizontal reinforcement shall be provided in the top of footings, at the top of wall openings, at structurally connected roof and floor levels, and at the top of parapet walls. Only horizontal reinforcement which is continuous in the wall shall be considered in computing the minimum area of reinforcement.

If the wall is constructed of more than two units in thickness, the minimum area of required reinforcement shall be equally divided into two layers, except where designed as retaining walls. Where reinforcement is added above the minimum requirements such additional reinforcement need not be so divided.

In bearing walls of every type of reinforced masonry there shall be not less than one 1/2 inch bar or two 3/8 inch bars on all sides of, and adjacent to, every opening which exceeds 24 inches in either direction, and such bars shall extend not less than 40 diameters, but in no case less than 24 inches beyond the corners of the opening. The bars required by this paragraph shall be in addition to the minimum reinforcement elsewhere required.

12.2.2 Cont.

When the reinforcement in bearing walls is designed, placed and anchored in position as for columns, the allowable stresses shall be as for columns. The length of the wall to be considered effective shall not exceed the center-to-center distance between loads nor shall it exceed the width of the bearing plus four times the wall thickness.

12.3 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any construction permitted in Chapter 12A.

12.4 SEISMIC PERFORMANCE CATEGORY B

Buildings assigned to Category B shall conform to all the requirements for Category A and to the additional requirements and limitations of this Section.

12.4.1 CONSTRUCTION LIMITATIONS

Masonry components shall be constructed to conform to the limitations of this Section.

(A) DESIGN. Structural and nonstructural components of the building shall be designed and reinforced as specified in Table 12.1. The letters designated A, B, and C in the Table refer to subsections (A), (B), and (C) of 12.2.2.

(B) TIES. In addition to the requirements of Sec. 12A.6.3(F), additional ties shall be provided around anchor bolts which are set in the top of a column or pilaster. Such ties shall engage the bolts and at least four vertical column bars for reinforced masonry. Such ties shall be located within the top 4 inches of the member and shall consist of not less than two No. 4 or three No. 3 ties.

(C) SHEAR WALLS. Shear walls shall conform to the applicable requirements of Sec. 12.7.

EXCEPTION:
The reinforcement provisions of Sec. 12.7.1 need not apply to reinforced masonry elements designed according to Sections 12.2.2(A) or (B).

(D) SCREEN WALLS. All screen walls shall be reinforced. Joint reinforcement shall be considered effective in resisting stresses. The units of a panel shall be so arranged that either the horizontal or the vertical joint containing reinforcing is continuous without offset. This continuous joint shall be reinforced with a minimum steel area of 0.03 square inch. Reinforcement shall be embedded in mortar or grout.

Joint reinforcing may be composed of two wires made with welded ladder or trussed wire cross ties. In calculating the resisting capacity of the system, compression and tension in the spaced wires may be utilized. Ladder wire reinforcing shall not be spliced and shall be the widest that the mortar joint will accommodate allowing 1/2 inch of mortar cover.

Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel.

(E) NONSTRUCTURAL COMPONENTS. Nonstructural walls, partitions, and components shall be designed to support themselves and to resist seismic forces induced by their own weight. Holes and openings shall be suitably stiffened and strengthened. Nonstructural walls and partitions shall be anchored in accordance with the requirements of Sec. 12A.2.6.

(F) CONSTRUCTION TYPE. Unreinforced cavity wall construction shall not be used for any structural masonry.

(G) GROUTED MASONRY--MULTIWYTHE. Grouted masonry is that form of construction made with brick or solid concrete units in which interior joints of masonry are filled by pouring grout therein as the work progresses. Only Type M or Type S mortar shall be used.

Toothing of masonry walls is permitted only when designed and detailed by the design engineer or architect and only at approved locations. Racking is to be held to a minimum.

12.4.1 Cont.

When reinforced in accordance with the following requirements it shall be classified as reinforced grouted masonry--multiwythe. All required reinforcement except masonry joint reinforcement and column ties shall be embedded in grout. All other reinforcement shall be embedded in mortar or grout. All vertical reinforcement shall be held firmly in place during grouting by a frame or suitable equivalent devices. All horizontal reinforcement in the grout space shall be tied to the vertical reinforcement or held in place during grouting by equivalent means.

(H) REQUIRED STRENGTHS FOR MORTAR AND GROUT. Mortar and grout strength shall conform to the requirements of Sec. 12A.8.2.

(I) JOINTS. All hollow units shall be laid with face shell bed joints and head joints filled solidly with mortar for a distance in from the face of the unit not less than the thickness of the face shells unless more stringent construction is required by this Chapter, Chapter 12A, or by design. Cross webs and end shells of all starter courses shall be bedded on mortar. This applies to units laid on foundations or floor slabs, and all courses of piers, columns, and pilasters.

Concrete abutting structural masonry such as at starter courses or at wall intersections not designed as true separation joints, shall be roughened to a full amplitude of 1/8 inch, and shall be bonded to the masonry per the requirements of this Chapter as if it were masonry. Unless keys are provided, vertical joints shall be considered to be stacked bond.

(J) GLASS MASONRY. Glass block shall be laid in Types M, S, or N mortar. Both vertical and horizontal mortar joints shall be at least 1/4 inch and not more than 3/8 inch thick and shall be completely filled.

Glass block panels shall have reinforcement in the horizontal mortar joints, extending from end to end of mortar joints, but not across expansion joints, with any unavoidable joints spliced by lapping the reinforcement not less than six (6) inches. The reinforcement shall be spaced not more than two (2) feet apart vertically. In addition, reinforcements shall be placed in the joint immediately below and above any openings within a panel. The reinforcement shall consist of two (2) parallel, longitudinal, galvanized steel wires, No. 9 gage or larger, spaced two (2) inches apart, and having welded thereto No. 14 or heavier gage cross wires at intervals not exceeding eight (8) inches, or the equivalent approved by the Regulatory Authority.

(K) REINFORCEMENT DEVELOPMENT, ANCHORAGE AND SPLICES. The requirements of 12A.6.3(D) are applicable except that calculated stress shall be replaced with yield strength. The following Subsections 1 and 2 replace subsections 5 and 7, respectively, in 12A.6.3(D).

1. Development Lengths. The basic development length, l_d, for deformed reinforcement shall be at least $.05\ d_b^2\ f_y/\sqrt{f_g}$ but not less than 24 d_b for reinforcement of 40,000 psi yield strength nor 36 d_b for reinforcement over 40,000 psi yield strength, nor less than 12 inches for reinforcing bars and 6 inches for masonry joint reinforcement where:

d_b = The diameter of the smaller bar spliced, inches.

f_y = The specified bar yield strength, psi.

f_g = The strength of the mortar or grout, as applicable, immediately surrounding the reinforcement but not more than the prism strength, psi.

Development lengths for plain reinforcing shall be twice that required for deformed reinforcement but not less than 12 inches.

EXCEPTIONS:
For deformed main compression reinforcement in columns that are not part of the seismic systems, these values may be reduced to 18 d_b for bars of 40,000 psi yield strength and 27 d_b for bars over 40,000 psi yield strength.
In flexural members that are not part of the primary lateral load resisting system the development lengths may be reduced where excess reinforcement is provided. For these cases, the previously determined development lengths may be multiplied by the ratio of the area of reinforcement required by design to that provided.

12.4.1 Cont.

2. Splices. Splices shall be made only at such points and in such manner that the strength of the member will not be reduced. Splices shall be made by lapping the bars, by welding, or by mechanical connections. Lapped splices shall not be used for tension tie members.

Lengths of laps, in inches, for deformed reinforcement shall be at least 0.08 $d_b^2 f_y/\sqrt{f_g}$ but not less than 40 d_b for reinforcement of 40,000 psi yield strength nor less than 60 d_b for reinforcement over 40,000 psi yield strength, nor less than 12 inches for reinforcing bars and 9 inches for masonry joint reinforcement. Lap lengths for plain reinforcing shall be twice that required for deformed bars but not less than 12 inches. The terms d_b, f_y, and f_g shall be as defined in Sec. 12.4.1(K)1.

EXCEPTION:
For deformed main compression reinforcement in columns that are not part of the seismic system, the lap length may be reduced to 30 d_b for bars of 40,000 psi yield strength and 45 d_b for bars over 40,000 psi yield strength.

Welded or mechanical connections shall develop the yield strength of the bar in tension.

EXCEPTION:
For compression bars in columns that are not part of the seismic system and are not subject to flexure the compressive strength need only be developed.

(L) DISTRIBUTION OF CONCENTRATED LOADS. Concentrated loads shall not be considered to be distributed by metal ties in stacked bond construction, nor to be distributed across continuous vertical joints. This provision shall apply when considering overturning effects in shear walls if stacked bond is not prohibited.

12.4.2 MATERIAL LIMITATIONS

The following materials shall not be used for any structural masonry:

Unburned Clay Masonry

Structural Clay Load Bearing Tile

Masonry Cement Mortar with Air Contents Greater than 15 percent

12.5 SEISMIC PERFORMANCE CATEGORY C

Buildings assigned to Category C shall conform to all of the requirements for Category B and to the additional requirements and limitations of this Section.

12.5.1 CONSTRUCTION LIMITATIONS

Masonry components shall be constructed to conform to the limitations of this Section.

(A) REINFORCEMENT. All masonry shall be reinforced masonry conforming to Section 12.2.2(C) except one-story residences of running bond construction located in map area 5 may conform to Section 12.2.2(B).

(B) TIE ANCHORAGES. In addition to the requirements of Sec. 12A.6.3.(D) for tie anchorages, a minimum turn of 135 degrees plus an extension of at least 6 tie diameters but not less than 4 inches at the free end of the tie shall be provided.

(C) REINFORCED COLUMNS. In addition to the requirements of Sec. 12A.6.3(F) for reinforced masonry columns, no longitudinal bar shall be farther than 6 inches from a laterally supported bar. Except at corner bars, ties providing lateral support may be in the form of cross-ties engaging bars at opposite sides of the column.

The tie spacing for the full height of masonry shear wall boundary columns and all other columns stressed by tensile or compressive axial overturning forces due to seismic effects and for the tops and bottoms of all other columns for a distance of 1/6 of clear column height but not less than 18 inches nor the maximum column dimension shall be not greater than 16 bar diameters nor

8 inches. Tie spacing for the remaining column height shall be not greater than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches.

(D) SHEAR WALL BOUNDARY ELEMENTS. Boundary members when used shall conform to one of the following:

1. Sec. 11.8.4 when of reinforced concrete or structural steel.

2. Sec. 12.5.1(C) when of masonry.

3. Sec. 12.7.2.

(E) JOINT REINFORCEMENT. Longitudinal masonry joint reinforcement may be used in reinforced grouted masonry and reinforced hollow unit masonry only to fulfill minimum reinforcement ratios but shall not be considered in the determination of the shear strength of the member.

(F) STACKED BOND CONSTRUCTION. The minimum ratio of horizontal reinforcement shall be 0.0015 for all structural walls of stacked bond construction. The maximum spacing of horizontal reinforcing shall not exceed 24 inches. Where reinforced hollow unit construction forms part of the seismic resisting system, the construction shall be grouted solid and all head joints shall be made solid through the use of open end units.

(G) WALLS. Every structural wall in reinforced masonry construction whose horizontal length is between 2 and 5 times its thickness or less than 1/2 the height of adjacent openings shall have all horizontal steel in the form of ties except that in walls less than 12 inches in nominal thickness and in reinforced grouted multi-wythe construction such steel may be in one layer in the form of hairpins.

(H) HOLLOW UNIT MASONRY. Hollow unit masonry construction, where certain cells are continuously filled with concrete or grout, and reinforcement, in accordance with 12.2.1(B)3, is embedded therein shall be classified as reinforced hollow unit masonry. Reinforced hollow unit masonry shall generally be one wythe in thickness. If constructed of more than one wythe, each wythe shall be designed as a separate element or wall or the wythes shall be bonded together by means approved by the Regualtory Agency. This bonding shall be designed so the wythes shall act as a unit.

Vertical cells to be filled shall have vertical alignment sufficient to maintain a clear, unobstructed continuous vertical cell measuring not less than 2 inches by 3 inches. If walls are battered or if alignment is offset, the 2 inch by 3 inch clear opening shall be maintained as measured from course to course.

(I) SHRINKAGE OF CONCRETE UNITS. Concrete masonry units used for structural purposes shall have a maximum linear shrinkage of 0.065 percent from the saturated to the oven-dry condition.

(J) CORE TESTS FOR SHEAR BOND IN GROUTED MASONRY-MULTITWYTHE. In addition to the requirements of Sec. 12A.8.3 the following provisions must be met for core tests for grouted masonry-multiwythe construction when such tests are required.

The unit shear strength shall not be less than 100 psi. Where an unusual number of cores fail during the cutting operation, the design authority shall determine if the test program is extensive enough to satisfy the requirements of Sec. 12A.1.5.

One test series shall be made for each 5,000 square feet of wall or equivalent but not less than one series for any building.

(K) MASONRY WALLS. Masonry wall thickness shall conform to Table 12A-1. The ratio of height or length to thickness of reinforced structural walls shall not exceed 25.

12.5.2 MATERIAL LIMITATIONS

The following materials shall not be used for any structural purpose:

Building Brick and Hollow Brick made from Clay or Shale of Grade NW

Hollow Load-bearing Concrete Masonry Units other than Grade N

12.5.2 Cont.

Type N Mortar

Masonry Cement

The following materials shall not be used for any nonstructural purpose:

Unburned Clay Masonry

Structural Clay Load-bearing and Nonload-bearing Wall Tile

Masonry Cement (Mortar with Air Content Greater than 15 Percent)

12.6 SEISMIC PERFORMANCE CATEGORY D

Buildings assigned to Category D shall conform to all of the requirements for Category C and to the additional requirements and limitations of this Section.

12.6.1 CONSTRUCTION LIMITATIONS

Materials for mortar and grout for structural masonry shall be measured in suitable calibrated devices. Shovel measurements are not acceptable. An approved admixture of a type that reduces early water loss and produces a net expansion action shall be used for grout for structural masonry unless it can be demonstrated that shrinkage cracks will not develop in the grout. The thickness of the grout between masonry units and reinforcing shall be a minimum of 1/2 inch for structural masonry.

(A) MINIMUM GROUT SPACE FOR GROUTED MASONRY. The minimum grout space for structural reinforced grouted masonry shall be 2-1/2 inches for low-lift construction and 3-1/2 inches for high-lift construction.

(B) REINFORCED HOLLOW UNIT MASONRY. Structural reinforced hollow unit masonry shall conform to requirements below:

1. Wythes and elements shall be at least 8 inches in nominal thickness with clear, unobstructed continuous vertical cells, without offsets, large enough to enclose a circle of at least 3-1/2 inches in diameter and with a minimum area of 15 square inches.

2. All grout shall be coarse grout. Grout consolidation shall be by mechanical vibration only. All grout shall be reconsolidated after excess moisture has been absorbed but before workability has been lost.

3. Vertical reinforcement shall be securely held in position at tops, bottoms, splices, and at intervals not exceeding 112 bar diameters. Approved intermediate centering clips or caging devices shall be used in high-lift construction, as required, to hold the vertical bars. Horizontal wall reinforcement shall be securely tied to the vertical reinforcement or held in place during grouting by equivalent means.

4. In wythes of less than 10 inch nominal thickness, in any vertical cell, there shall be a maximum of one No. 10 bar or two No. 8 bars with splices staggered for the two-bar situation.

5. The first exception of Sec. 12A.6.3(F) shall not apply; minimum nominal column dimension shall be 12 inches.

(C) STACKED BOND CONSTRUCTION. All stacked bond construction shall conform to the following requirements:

1. The minimum ratio of horizontal reinforcement shall be 0.0015 for nonstructural masonry and 0.0025 for structural masonry. The maximum spacing of horizontal reinforcing shall not exceed 24 inches for nonstructural masonry nor 16 inches for structual masonry.

2. Reinforced hollow unit construction which is part of the seismic resisting system shall (1) be grouted solid, (2) use double open end (H block) units so that all head joints are made solid, and (3) use bond beam units to facilitate the flow of grout.

12.6.1 Cont.

3. Other reinforced hollow unit construction used structurally, but not part of the seismic resisting system, shall be grouted solid and all head joints shall be made solid by the use of open end units.

12.6.2 MATERIAL LIMITATIONS

Hollow non-loadbearing concrete masonry units shall not be used. Sand-lime building brick, Building Brick and Hollow Brick made from Clay or Shale of Grade NW and Building Brick and Solid Loadbearing Concrete Masonry Units other than Grade N shall not be used for any structual masonry.

12.6.3 SPECIAL INSPECTION

Special inspection shall be provided for all structural masonry.

12.7 SHEAR WALL REQUIREMENTS

Shear walls shall comply with the requirements of this Section.

12.7.1 REINFORCEMENT

The following reinforcement requirements apply to shear walls required to comply with the provisions of 12.2.2(C).

The minimum ratio of reinforcement for shear walls shall be 0.0015 in each direction. The maximum spacing of reinforcement in each direction shall be the smaller of the following dimensions: one-third the length and height of the element but not more than 48 inches. The area and spacing of reinforcement perpendicular to the shear reinforcement shall be at least equal to that of the required shear reinforcement. The portion of the reinforcement required to resist shear shall be uniformly distributed.

EXCEPTION:
For shear walls constructed using running bond, the ratio of reinforcement may be decreased to 0.0007 provided that all shear is resisted by the reinforcement. The sum of the ratios of horizontal and vertical reinforcement shall not be less than 0.002.

Reinforcement required to resist wall shear shall be terminated with a standard hook which terminates beyond the boundary reinforcing at the end of the wall sections. The hook may be turned up, down, or horizontally and shall be embedded in mortar or grout. Wall reinforcement terminating in boundary columns or beams shall be fully anchored into the boundary elements.

12.7.2 BOUNDARY MEMBERS

Where cross walls or boundary members form a part of the shear wall system, the intersection shall be constructed as required for the walls themselves. Connections to concrete shall conform to Sec. 12A.2.1. Where the boundary members are of structural steel, the shear transfer between the wall and the boundary member shall be developed by fully encasing the element in grout, by dowels, bolts, or shear lugs, or by similar approved methods.

When the structural system, as described in Chapter 3 and Table 3-B, consists of substantially complete vertical load-carrying frame, boundary members shall be provided at each end of the wall. The members shall be of the same construction as the frame columns. Where the frame is a special moment frame, those columns shall conform to the requirements for such members in Chapters 10 and 11. Also see Sec. 12.5.1(D) for Category C and D.

The required vertical boundary members and such other similar vertical elements as may be required shall be designed to carry all the vertical forces resulting from the wall loads, the tributary dead and live loads, and the seismic forces prescribed in these provisions.

Horizontal reinforcing in the walls shall be anchored to the vertical elements. Where the boundary element is structural steel this shall be accomplished by welding or by extension, with bends if required, into grout fully surrounding the column.

12.7.3 STRESSES

For loading combinations including in-plane seismic forces, allowable compression stresses at any point shall not exceed those allowed for axial compression. For unreinforced masonry designed by Sec. 12A.6.1, the allowable working stress values are given in Table 12A-3. The allowable working stress values for reinforced masonry shall be the allowable working stresess given in Table 12A-5 and applicable reductions for slenderness effects shall apply. The minimum horizontal distance between lateral supports may be considered for walls as well as the minimum vertical distance. Formula 12A-9 shall not be used.

Vertical stresses in shear walls shall be determined from the combined effects of vertical load and from the overturning effects of lateral loads. Minimum vertical loads shall be considered. Formula 3-2a shall be used for unreinforced masonry design.

In computing the shear resistance of the wall, only the web shall be considered. For unreinforced masonry the depth of the web may be considered out to out of flanges.

EXEPTION:
For pier type wall elements that do not extend from floor to floor compression stresses under combined loading at any point may be limited to those allowed for flexural compression provided that Formula 12A-9 is also satisfied.

12.7.4 HORIZONTAL COMPONENTS

When shear reinforcing is required for loads that include seismic effects and diagonal bars conforming to Sec. 12A.6.4(D) are not provided, reinforcement approximately perpendicular to the required shear reinforcement shall be provided equal in amount and spaced not further apart than is required for the shear reinforcing. Horizontal reinforcing shall anchor into or be continuous through the pier elements. Horizontal components may be separated from the shear wall system by means of joints. The joints shall provide for building movement determined in accordance with Sec. 3.8. The horizontal components shall be anchored to the building and designed as otherwise required by these provisions.

TABLE 12.1

DESIGN AND REINFORCEMENT REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORY B

Type of Construction	Map Area 2		Map Area 3		Map Area 4	
	Buildings under 35 ft	Buildings over 35 ft	Buildings under 35 ft	Buildings over 35 ft	Buildings under 35 ft	Buildings over 35 ft
Structural Components						
Running Bond	A	B	B	B	B	C
Stacked Bond	B	B	C	C	C	C
Nonstructural Components						
Running Bond	A	A	A	A	B	B
Stacked Bond	A	A	A	B	B	B

Note: 1. The letters A, B, and C refer to subsections (A), (B), and (C) of 12.2.2.
2. Map areas refer to figure 1.2 (in the Applied Technology Council prepared report Tentative Provisions for the Development of Seismic Regulations for Buildings, National Bureau of Standards Special Publication 510, Washington, D.C., 1978).

PART B - AMENDMENTS TO CHAPTER 12A

Listing of Amendments to Chapter 12A

ection with Modification[1]			BSSC Ballot Number
12A.1	GENERAL		NC[2]
12A.1.1	DEFINITIONS:	AREA, CROSS-SECTIONAL through SHOVED JOINT	5A/1
		LOAD BEARING	5A/2-ATC
		MASONRY through STRUCTURAL	5A/3-5A/8
12A.1.2	REFERENCE DOCUMENTS		5A/9
12A.1.3	SYMBOLS		5A/10
12A.1.4 - 12A.1.6	CRITERIA FOR MASONRY UNITS MASONRY UNIT SURFACES FOR GROUTED MASONRY		5A/11
12A.1.7 - 12A.1.10	RE-USE OF MASONRY UNITS GLASS BUILDING UNITS		5A/12
12A.1.11 - 12A.1.12	GLAZED AND PREFACED UNITS WATER		5A/13
12A.1.14	CEMENT		5A/14-ATC[3]
12A.1.15 - 12A.1.17(B)	LIME TYPE		5A/15
12A.1.17(C)	CONSISTENCY		5A/16-ATC
12A.1.17(D)	ADMIXTURES		5A/16
12A.1.17(E)	MEASURING AND MIXING		5A/17-ATC
12A.1.17(F) - 12A.1.17(G)	STRENGTH ALUMINUM EQUIPMENT		5A/18
12A.2	CONSTRUCTION		5A/18
12A.2.1	JOINTS		5A/18, 5A/19-ATC, 5A/20, 5A/21
12A.2.2	BOND PATTERN		5A/22
12A.2.3 - 12A.2.6	COREBELING ANCHORAGE		5A/22
12A.2.7	BOLT PLACEMENT		5A/22, 5A/23-ATC, 5A/24-ATC
12A.2.8 - 12A.3.4	PENETRATIONS AND EMBEDMENTS CAVITY WALL MASONRY		5A/25
12A.3.5	GROUTED MASONRY - MULTIWYTHE WALLS		5A/26-ATC, 5A/27, 5A/28
	(A) LOW LIFT		5A/28

[1] Sections are listed to the extent necessary to identify ballot numbers.

[2] NC indicates no change proposed from original ATC.

[3] Used original ATC wording to resolve an item that did not pass the ballot.

Section with Modification		BSSC Ballot Number
	(B) HIGH LIFT	5A/28, 5A/29-ATC, 5A/30
	(C) REINFORCED CONSTRUCTION	5A/31-ATC, 5A/32
12A.3.6	HOLLOW UNIT MASONRY	5A/32, 5A/33-ATC
	(A) GROUTING PROCEDURES	5A/34-ATC, 5A/35, 5A/36-ATC, 5A/37-ATC 5A/38
12A.3.7	GLASS MASONRY	5A/39
12A.4	DETAILED REQUIREMENTS	5A/40
12A.4.1 - 12A.4.2	COMBINATION OF DISSIMILAR UNITS THICKNESS OF WALLS	5A/41
12A.4.3 - 12A.6.1	CHASES AND RECESSES DESIGN PROCEDURE FOR UNREINFORCED MASONRY	5A/42
12A.6.2	ALTERNATE DESIGN PROCEDURES FOR UNREINFORCED MASONRY	5A/43
	(A) UNREINFORCED BRICK MASONRY USING SOLID CLAY UNITS	5A/43
	Items 1-5a	5A/43
	Items 5b, c, and d	5A/44-ATC
	Items 6-12	5A/45
	(B) UNREINFORCED CONCRETE MASONRY	5A/46
	Item 1	5A/47-ATC
	Items 2-4	5A/48-C[4]
	(C) DESIGN, UNREINFORCED HOLLOW CLAY MASONRY	5A/49-ATC, 5A/50-ATC, 5A/51
	(C)1.b SHEAR WALLS WITH NO SHEAR REINFORCEMENT	5A/52[5]
	(C)1.c SHEAR WALL OVERTURNING	5A/53
12A.6.3	DESIGN PROCEDURE FOR REINFORCED MASONRY	5A/54, 5A/55, 5A/56, 5A/57-ATC, 5A/58-ATC, 5A/59, 5A/60-ATC-C,[6] 5A/61-ATC
12A.6.3(F)	REINFORCED MASONRY COLUMNS	5A/62-ATC, 5A/63
12A.6.4	MASONRY SHEAR WALLS - Items A-C	5A/65
	(D) WALL SHEAR	5A/66-ATC, 5A/67
	(E) BOUNDARY ELEMENTS	5A/63-ATC

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[4] Indicates the ballot item passed but was modified slightly for internal consistency.

[5] This section needs future study for review of the allowables.

[6] Used a modification of the original ATC wording to resolve an item that did not pass the ballot.

Section with Modification		BSSC Ballot Number
12A.6.5	SCREEN WALLS	5A/67
12A.7	INSPECTIONS AND TESTS	5A/67
12A.7.1	FREQUENCY OF INSPECTIONS AND TESTS	5A/67
12A.7.2	SPECIAL INSPECTION AND TESTS	
	(A) SPECIAL INSPECTION	5A/67
	(B) TESTS AND/OR CERTIFICATIONS	5A/68-ATC, 5A/69, 5A/70, 5A/71
12A.7.3	LOAD TESTS	5A/71
12A.7.4	REPORTING	5A/71
12A.8	TEST CRITERIA	5A/71
12A.8.1	MASONRY PRISMS	5A/71
12A.8.2	GROUT TEST AND FILL MORTAR TESTS	5A/71, 5A/72-ATC
12A.8.3	CORE TESTS FOR SHEAR BOND	5A/73
Table 12A-1	MAXIMUM RATIO OF HEIGHT TO THICKNESS AND MINIMUM THICKNESS OF MASONRY WALLS	5A/74-ATC, 5A/75-ATC, 5A/76-ATC, 5A/77-ATC
Table 12A-2	ALLOWABLE WORKING STRESSES IN MASONRY WALLS	5A/77
Table 12A-3	ASSUMED COMPRESSIVE STRENGTH OF MASONRY	5A/77
Table 12A-4	ALLOWABLE WORKING STRESSES FOR REINFORCED MASONRY	5A/78-ATC, 5A/79
Table 12A-5	ALLOWABLE SHEAR ON BOLTS	5A/80-ATC, 5A/81

The following ballot items included deletions: 5A/20, 5A/27

CHAPTER 12A

REFERENCE GUIDELINE FOR MASONRY CONSTRUCTION

Sec. 12A.1 GENERAL

This Chapter applies to new masonry construction of a structural and nonstructural nature. It is included because a nationally applicable seismic design standard is not available. Except as portions of it may be incorporated by reference, it does not apply to the repair or rehabilitation of existing masonry nor to the construction of masonry veneers. See Chapter 13 and 14 for repair and Chapter 8 for veneers.

12A.1.1 DEFINITIONS

The following definitions and those of Chapter 2 provide the meaning of terms used in this Chapter.

AREA, GROSS CROSS-SECTIONAL. The total area face-to-face of masonry including cells or cavities of a section perpendicular to the direction of loading. Reentrant spaces are excluded in the gross area unless these spaces are to be occupied by masonry portions of adjacent units.

AREA, NET. The gross cross-sectional area at any plane minus the area of ungrouted cores, notches, cells, etc. Net area is the actual surface area of a cross-section.

AREA, NET BEDDED. The actual area of masonry units that bear on the mortar bed with deductions for rakes and similar joint treatments. In grouted construction the continuous vertical filled grout cores or grout spaces are included.

AREA, NET CROSS-SECTIONAL OF HOLLOW UNIT. The gross cross-section area of a section minus the average area of ungrouted cores or cellular and other spaces.

AREA, NET VERTICAL SHEAR. The minimum gross cross-sectional area at any vertical plane of hollow units less their ungrouted cores or the mortar contact areas at head joints, whichever is less.

BOND, RUNNING. When in a wythe, at least 75 percent of the units in any transverse vertical plane lap the ends of the units above and below a distance not less than 1.5 inch or one-half the height of the units, whichever is greater; the wythe, for the purpose of this document, shall be considered to be laid in running bond. (Note that for the purpose of this definition center bond or half bond is not necessarily required to obtain running bond.) Where corners and wall intersections are constructed in a similar fashion, they shall be considered to be laid in running bond.

BOND, STACKED. All conditions of head joints not qualifying as running bond and all continuous vertical joints (excepting true joints such as expansion and contraction joints) shall be considered to be stacked bond construction.

DIMENSIONS. Overall dimensions given for masonry units and walls are nominal; actual dimensions of unit masonry may not be decreased by more than 1/2 inch from the nominal dimension. Dimensions of grout spaces, clearances and cover given are actual.

EFFECTIVE ECCENTRICITY. The actual eccentricity of the applied vertical load including that caused by member deflections and thermal or other movements of connected members plus the additional eccentricity which would produce a moment equal in magnitude to that produced by the lateral loads.

GROUTED MASONRY. Masonry composed of hollow units in which designated cells are solidly filled with grout or masonry of two or more wythes in which the cavities between wythes are solidly filled with grout.

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

JOINT COLLAR. The vertical space separating a wythe of masonry from another wythe or from another continuous material and filled with mortar or grout.

JOINT, HEAD. The vertical mortar joint between ends of masonry units.

12A.1.1 Cont.

JOINT, SHOVED. Produced by placing a masonry unit on a mortar bed and then immediately shoving it a fraction of an inch horizontally against the mortar in the head joints to effect solid, tight joints.

LOAD BEARING. Synonymous with Structural.

MASONRY. An assemblage of masonry units bonded together with mortar or grout.

(A) MASONRY, REINFORCED. Masonry in which reinforcement is used to resist forces as well as the purpose of crack control. See Section 12.2 for allowable stresses.

(B) MASONRY, UNREINFORCED. Masonry in which reinforcement is used only for the purpose of crack control. See Section 12.2 for allowable stresses.

MASONRY UNIT. Any brick, tile, stone, or block conforming to the requirements specified in this Chapter.

NONSTRUCTURAL. This term refers to components or systems which do not serve in providing resistance to loads or forces other than induced by their own weight. Walls that enclose a building or structure's interior are structural components.

REINFORCEMENT RATIO. This is the ratio of the areas of reinforcement to the gross cross-sectional area of the masonry perpendicular to the reinforcement.

SHEAR WALL. A vertical component resisting lateral forces by in-place shear and flexure.

STRUCTURAL. This term refers to a system or component which serves in providing resistance to loads or forces other than induced by the weight of the element itself. All portions of the seismic resisting system are structural, but not all structural components need be part of the seismic resisting system. Bracing components, bracing systems, and walls that enclose a building or structure's exterior are structural elements.

12A.1.2 REFERENCE DOCUMENTS

The following standards apply to masonry materials and to the testing thereof:

MATERIALS AND TESTING	STANDARD DESIGNATION
<u>Building and Facing Brick</u>	
Clay and Shale	ASTM C62, C216, C652*
Sand-Lime	ASTM C73
Method of Test	ASTM C67
<u>Concrete Masonry Units</u>	
Hollow Load-Bearing	ASTM C90
Solid Load-Bearing	ASTM C145
Hollow Nonload-Bearing	ASTM C129
Brick	ASTM C55
Method of Test	ASTM C140
<u>Structural Clay Tile</u>	
For Walls - Load-Bearing	ASTM C34, C212, C126
For Walls - Nonbearing	ASTM C56
For Floors	ASTM C57
<u>Cast Stone</u>	ACI 704
<u>Unburned Clay</u>	Uniform Building Code Standard 24-14

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* And Western States Clay Products Standard Specifications for Hollow Brick.

12A.1.2 Cont.

MATERIALS AND TESTING	STANDARD DESIGNATION
Reinforcement	
Reinforcing Steel	ASTM A615, A616, A617, and A706
Masonry Joint Reinforcement	ASTM A82
Welding	AWS D12.1
Cement	
Blended Hydraulic Cement	ASTM C595
Portland Cement and Air-Entraining Portland Cement	ASTM C150
Masonry Cement	ASTM C91
Lime	
Quicklime	ASTM C5
Hydrated Lime for Masonry Purposes	ASTM C207
Processed Pulverized Quicklime	ASTM C51
Mortar	
Other than Gypsum	ASTM C270
Aggregates for Mortar	ASTM C144
Field Tests for Mortar	Sec. 12A.8.2
Grout	
Aggregates for Grout	ASTM C404
Field Tests for Grout	Sec. 12A.8.2
Masonry Assemblies	Sec. 12A.7 and 12A.8

12A.1.3 SYMBOLS

The symbols used in this Chapter are defined as follows:

a = Angle between inclined web bars and axis of the beam.

A_g = Gross cross-sectional area, square inches.

A_s = Effective cross-sectional area of reinforcement in a column or flexural member.

A_v = Total area of web reinforcement in tension within a distance of s, or the total area of all bars bent up in any one plane, square inches.

b = Effective width of rectangular section or stem of I- or T-sections, inches.

C_e = Eccentricity coefficient.

C_s = Slenderness coefficient.

d = Effective depth from compression face of beam or slab to centroid of longitudinal tensile reinforcement, inches.

d_b = Reinforcement diameter, inches.

e = Effective eccentricity, inches.

e_i = Effective eccentricity about the principal axis which is normal to the length of the element.

12A1.3 Cont.

e_1 = Smaller effective eccentricity at lateral support at ends of member (at either top or bottom), inches.

e_2 = Larger effective eccentricity at lateral support at ends of member (at either top or bottom), inches.

e_t = Effective eccentricity about the principal axis which is normal to the thickness of the element.

E_m = Modulus of elasticity of masonry in compression, psi.

E_s = Modulus of elasticity of steel in tension or compression, psi.

f_g = The strength of the mortar or grout, as applicable, immediately surrounding the reinforcement but not more than the prism strength, psi (see 12.4.1(K)).

f_m = Allowable compressive unit stress, psi.

f'_m = Compressive strength of masonry, psi.

f'_{mb} = Brick masonry design strength, psi.

f_s = Allowable stress in reinforcement, psi.

f_t = Allowable flexural tensile stress in masonry, psi.

f_v = Allowable unit stress in web reinforcement, psi.

h = Effective height, the height or length of a column or wall used for purposes of determining slenderness effects.

i = Effective length of rectangular wall element or column.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

l_a = A dimension determined in accordance with Sec. 12A.6.3(D), inches.

l_d = Development length, inches.

M_c = Minimum allowable moment capacity, inch-pounds.

n = Ratio of modulus of elasticity of steel to that of masonry.

$$n = \frac{E_s}{E_m}$$

ρ = A_s/bd, ratio of the area of reinforcement to the area (bd).

P = Allowable vertical load, pounds.

r = Radius of gyration, inches.

R_e = Eccentricity ratio for elements subject to bending about both principal axes.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement, inches.

t = Effective thickness, inches.

v = Shearing unit stress, psi.

v_m = Allowable unit shearing stress in the masonry, psi.

V = Total shear, pounds.

β_b = A ratio as determined by Sec. 12A.6.3(D)1.

12A.1.4 CRITERIA FOR MASONRY UNITS

Masonry units shall be of a type, quality, and grade consistent with the applicable provisions and intent of the referenced documents considering:

The intended usage such as structural or nonstructural.

The surrounding environment such as severe frost action in presence of water, contact with the ground, exposure to the weather, and/or enclosure within a building.

Type, quality, grade, and any similar additional special requirements of this Chapter or Chapter 12 for masonry units, all as applicable, shall be indicated on the design documents.

12A.1.5 INITIAL RATE OF ABSORPTION

At the time of laying, burned clay units and sand-lime units shall have a rate of absorption not exceeding 0.025 ounce per square inch during a period of one minute. Test procedures shall be in accordance with ASTM C67-73. In the absorption test the surface of the unit shall be held 1/8 inch below the surface of the water. Water content shall be that of the units to be laid, i.e., the units shall not be dried.

12A.1.6 MASONRY UNIT SURFACES FOR GROUTED MASONRY

Units for grouted masonry shall have all surfaces to which grout is to be applied capable of adhering to grout with sufficient tenacity to resist the required shearing stress. Tests, when required, shall conform to Sec. 12A.7 and 12A.8.3.

12A.1.7 RE-USE OF MASONRY UNITS

Masonry units may be reused when clean, whole, and in conformance with the requirements of this Chapter and those of the applicable reference documents. Conformance must be established by tests of representative samples.

12A.1.8 CAST STONE

Every cast stone unit more than 18 inches in any dimension shall conform to the requirements for concrete in Chapter 11.

12A.1.9 NATURAL STONE

Natural stone shall be sound, clean, and in conformity with other provisions of this Chapter.

12A.1.10 GLASS BUILDING UNITS

Glass block shall have unglazed or satisfactorily treated surfaces to allow adhesion on all mortared faces. Units shall be constructed so that a minimum panel thickness of 3.0 inches can be obtained at the mortar joints.

12A.1.11 GLAZED AND PREFACED UNITS

Glazed and prefaced units shall conform to the physical criteria for unglazed and unfaced units in addition to any special requirements desired for the exposed finish. Surfaces receiving mortar and surfaces to be grouted shall be unglazed.

12A.1.12 WATER

Water used in mortar, grout, or masonry work shall be clean and free from injurious amounts of oil, acid, alkali, organic matter, or other harmful substances.

12A.1.14 CEMENT

Cements for mortar are limited to those allowed by ASTM C270, this Chapter and Chapter 12.

EXCEPTION:
Approved types of plasticizing agents may be added to portland cement Type I or Type II in the manufacturing process, but not in excess of 12 percent of the total volume. Plastic or water-

12A.1.14 Cont.

proofed cements so manufactured shall meet the requirements for portland cement except in respect to the limitations on insoluble residue, air-entrainment, and additions subsequent to calcination.

Cements for grout shall by Type I, IA, II, IIA, III, IIIA, or V portland cement or Type 1S, 1S-A, 1S (MS), 1S-A(MS), 1P, or 1P-A blended hydraulic cement. Except masonry cement shall not be used for grout.

12A.1.15 LIME

Lime for mortar and grout is limited to those allowed by ASTM C207.

12A.1.16 MORTAR

Mortar shall be prepared in accordance with either the property or porportions procedures given in ASTM C270.

Where mortar colors are used or where minimum compressive strengths are required for mortar used in the work, only the Property Specifications shall be used. Field tests shall conform to Sec. 12A.7 and 12A.8.2

Where the source or the proportions of ingredients for mortar, classified in accordance with the Property Specifications, are intended to be changed during the course of the work, acceptability of the new mortar shall be reestablished in accordance with ASTM C270.

ASTM C270 Type O and K mortar shall not be used.

Admixtures shall be added only after approval by the Regulatory Agency. Coloring ingredients shall be limited to inert mineral or inorganic synthetic compounds not exceeding 15 percent of the weight of cement or carbon black not exceeding 3 percent of the weight of cement.

To maintain plasticiity, mortar may be retempered with water by the method of forming a basin in the mortar and reworking it. However, any mortar which has hardened or stiffened due to hydration of the cement shall not be used.

12A.1.17 GROUT

(A) PROPORTIONING. Grout shall be proportioned by volume and shall have sufficient water added to produce consistency for pouring without segregation. Aggregates shall conform to ASTM C404 except that larger size coarse aggregate may be used in large grout spaces where approved by the Regulatory Agency.

EXCEPTION:
Grout may be proportioned by weight when weight-volume relationships are established and periodically verified.

(B) TYPE. The requirements for coarse and fine grout shall be as follows:

1. Fine Grout. Fine grout shall be composed by volume, of one part cement, to which may be added not more than 1/10 part hydrated lime and 2-1/4 to 3 parts of sand.

2. Coarse Grout. Coarse grout shall be composed by volume, of one part of cement, to which may be added not more than 1/10 part hydrated lime, two to three parts sand, and one to two parts gravel. Larger proportions of gravel may be used in large grout spaces where approved by the Regulatory Agency.

(C) CONSISTENCY. Grout shall have a consistency, considering the methods of consolidation to be utilized, to completely fill all spaces to be grouted without segregation except that slumps shall not be less than 4.5 inches for all grout nor more than 10 inches for fine grout or 9 inches for coarse grout.

(D) ADMIXTURES. Admixtures shall be approved by the Regulatory Agency.

12A.1.17 Cont.

(E) MEASURING AND MIXING. Materials for grout shall be measured in suitable calibrated devices. After the addition of water, all materials shall be mixed for at least three minutes in a drum-type batch mixer. Mixing equipment and procedures shall produce grout with the uniformity required for concrete by ASTM C94.

(F) STRENGTH. Grout shall attain the minimum compressive strength required by design or required to obtain the prism strength required by design, but shall not be less than 2000 pounds per square inch at 28 days. The Regulatory Agency may require field tests to verify the grout strength. Such tests shall be made in accordance with Sec. 12A.7 and 12A.8.2.

(G) ALUMINUM EQUIPMENT. Grout shall not be handled nor pumped utilizing aluminum equipment.

12A.1.18 REINFORCEMENT

Reinforcement over 1/4 inch (No. 2) in diameter shall be deformed bars.

Sec. 12A.2 CONSTRUCTION

Storage, handling, and preparation at the site shall conform to the following requirements.

Masonry materials shall be stored so that at the time of laying the materials are clean and not damaged.

Concrete masonry units shall not be wetted unless otherwise approved.

12A.2.1 JOINTS

All units shall be laid with shoved mortar joints. Solid units have all head and bed joints solidly filled unless otherwise approved.

All hollow units shall be laid with face shell bed joints and head joints filled solidly with mortar for a distance in from the face of the unit not less than the thickness of the face shells unless more stringent construction is required by this Chapter, Chapter 12, or by design. Cross webs and end shells of all starter courses shall be bedded on mortar. This applies to units laid on foundations or floor slabs or similar, and all courses of piers, columns, and pilasters.

Surfaces in contact with mortar or grout shall be clean and free of laitance, debris, or other deleterious materials.

Except as provided for firebrick or otherwise restricted, initial bed joint thickness shall not be less than 1/4 inch nor more than 1 inch; subsequent bed joints shall not be less than 1/4 inch and not more than 5/8 inch in thickness.

12A.2.2 BOND PATTERN

All bed joints shall be horizontal and all head joints between adjacent units shall be vertical.

EXCEPTIONS:
1. Rubble stone masonry joints may vary from the horizontal or vertical.
2. The joints in arches and similar construction may vary from the horizontal or vertical.
3. The joints in other masonry construction may vary from the horizontal or vertical provided the construction is approved in accordance with Sec. 1.5.

(A) REQUIREMENTS. Adjacent wythes shall be bonded to each other in accordance with the applicable provisions of Sec. 12A.3.

All wythes of all masonry walls and all corners and wall intersections shall be laid in running bond except where the true joints such as expansion and contraction joints are provided and except as follows.

Where not prohibited in Chapter 12 or this Chapter, unreinforced stacked bond masonry may be used with one of the mechanical bonding devices indicated in Sec. 12A.2.2(A)1, 2, and 3 below:

12A.2.2 Cont.

1. Not less than two continuous corrosion-protected wires conforming to ASTM A82 in bed joints spaced not over 16 inches vertically. The wires shall provide a minimum reinforcement ratio of 0.00027 or each shall have a minimum cross-sectional area of 0.017 square inch, whichever is greater. At corners and intersections the wires shall be bent and shall be continued beyond the bend. No splices of continuous wires shall occur within 12 inches of the bend. Splices of the continuous wires shall be at least 12 inches in length and splices of alternate wires shall be staggered.

2. Where only the corner or intersecting joints are of stacked bond construction these joints may be bonded by 1/4 inch diameter steel rods, bent into a rectangular shape so that two legs cross the joint, laid in bed joints spaced not over 16 inches vertically. The rods shall extend a distance equal to the length of the masonry units, but not less than 6 inches, beyond each side of the joint. For masonry construction with other than hollow units, corrosion-protected steel straps having the same total area may be used in lieu of the rods. The ends of the straps shall be bent up 2 inches or cross pins for anchorage shall be provided.

For brick masonry designed in accordance with Sec. 12A.6.2 where the intersecting walls are regularly toothed or blocked with 8 inch maximum offsets, the bonding may be provided with metal anchors. The anchors shall be 1/4 inch by 1-1/2 inches with ends bent up at least 2 inches, or with cross pins to form anchorage. Such anchors shall be at least 24 inches long, and shall be placed in bed joints spaced not over 48 inches vertically.

For nonstructural masonry the mechanical bond at intersecting joints, when required, shall be provided by corrosion-protected steel ties or clips at least 7/8 inch wide and not less than 16 gage or their wire equivalent, embedded in the bed joints, extending 3 inches minimum each side of the continuous vertical joint, placed not over 32 inches vertically.

3. For cavity walls the provisions of 1 and 2 above apply to each wythe.

12A.2.3 CORBELING

The slope of corbeling (angle measured from the horizontal to the face of the corbeled surface) shall not be less than 60°. The maximum horizontal projection of the corbel from the plane of the wall shall not exceed one-half the wythe thickness for cavity walls or one-half the wall thickness for all other walls.

12A.2.4 REINFORCEMENT

Reinforcement shall conform to the requirements of this Section. All metal reinforcement shall be free from loose rust and other coatings that would reduce bond to the reinforcement.

(A) BAR SPACING. The minimum clear distance between parallel reinforcement, except in columns, shall be not less than the reinforcement diameter nor 1 inch except that lapped splices may be wired together. The center-to-center spacing of bars within a column shall not be less than 2-1/2 times the bar diameter. In addition to the preceding, the minimum clear distance between parallel reinforcement embedded in coarse grout shall not be less than 1-1/3 times the maximum aggregate size.

(B) SPLICES. Splices in reinforcement may be made only at approved locations or as indicated on the approved design documents. Splices shall conform to the provisions of Sec. 12A.6.3(D)7.

(C) EMBEDMENT AND COVERAGE. All reinforcement shall be completely embedded in mortar or grout. Joint reinforcement embedded in mortar joints shall have not less than 5/8 inch mortar coverage from an exposed face and 1/2 inch from other faces. All other reinforcement shall have a minimum masonry coverage of one bar diameter, but not less than 3/4 inch except where exposed to water, weather, or soil in which case the minimum coverage shall be 2 inches. See Sec. 12A.3.5(C) and 12A.3.6(A) for minimum grout coverage.

(D) SIZE LIMITATIONS. Longitudinal wall bars and other longitudinal bars shall be limited to deformed bars, #3 minimum and #10 maximum, when used in reinforced or partially reinforced masonry construction.

12A.2.4 Cont.

EXCEPTIONS:

1. Number 11 bars may be used provided the grout cover, measured for masonry units to reinforcing bar, including areas at splices is at least 1-1/2 inches.
2. The size limits do not apply to masonry joint reinforcement or column ties. See Sec. 12A.6.3(E)2 and 12A.6.3(F)2.

(E) WELDING. Welding of reinforcement shall conform to AWS D12.1. Reinforcement to be welded shall conform to the chemical requirements of ASTM A706 or the chemical constituents shall be verified.

12A.2.5 TEMPERATURE LIMITATIONS

Cold weather construction shall conform to the requirements of "Recommended Practices and Guide Specifications for Cold Weather Construction," by the International Masonry Industry All-Weather Council.

When the ambient air has a temperature of more than 90°F in the shade, and has a relative humidity of less than 50 percent, protect newly erected masonry from direct exposure to wind and sun for 48 hours after installation.

12A.2.6 ANCHORAGE

Masonry walls shall be anchored to components providing lateral support as required by Sec. 3.7.6. Nonstructural walls required to be separated from the structural system shall be provided with anchorages which will permit relative movement between the wall and the structure as required by Sect. 3.8.

12A.2.7 BOLT PLACEMENT

Edge distances and center-to-center spacings shall not be less than required by Table 12A-6.

In grouted construction, all bolts shall be grouted in place. The bolts shall be accurately set with templates or by approved equivalent means and held in place to prevent movement. Grout coverage shall be as required for reinforcing bars of equivalent size.

In ungrouted construction, bolts shall be securely embedded in mortar except that for hollow unit masonry the cells containing bolts shall be grouted or mortared solid. There shall be at least 1/4 inch mortar between the bolts and masonry units for bolts set in mortar.

In cavity wall construction the wall shall be made solid at bolts for at least six diameters each side of the bolt.

Vertical bolts at the top of and near the ends of reinforced masonry walls shall be set within hairpins or ties located within 2.5 inches from the top of the wall. See Sec. 12A.6.3(F) and 12.4.1(B) for bolts at the top of piers, pilasters, and columns.

12A.2.8 PENETRATIONS AND EMBEDMENTS

No conduits, plumbing, and similar embedments, holes, sleeves, chases, recesses, or other weakening construction are permitted unless indicated on the approved plans. See Sec. 12A.4.4 and 12A.4.5.

12A.2.9 SUPPORT BY WOOD

Wood members shall not be used to support any permanent loads imposed by masonry construction except as provided in Sec. 9.5.2.

Sec. 12A.3 TYPES OF CONSTRUCTION

The types of masonry construction in Sec. 12A.3.1 through 12A.3.7 may be used for structural or nonstructural purposes and the type of masonry construction in Sec. 12A.3.8 may be used for nonstructural purposes subject to requirements of Chapter 12 and this Chapter.

12A.3.1 UNBURNED CLAY MASONRY

Unburned clay masonry is that form of construction made with unburned clay stablized with emulsified asphalt. Such units shall not be used in any building more than one story in height. All footing walls which support masonry of unburned clay units shall extend to an elevation not less than 6 inches above the adjacent ground at all points.

12.A.3.2 STONE MASONRY

Stone masonry is that form of construction made with natural or cast stone with all joints thoroughly filled.

In ashlar masonry, bond stones uniformly distributed shall be provided to the extent of not less than 10 percent of the area exposed faces.

Rubble stone masonry 24 inches or less in thickness shall have bond stones with a maximum spacing of 3 feet vertically and 3 feet horizontally, and if the masonry is of greater thickness than 24 inches, shall have one bond stone for each 6 square feet of wall surface on both sides.

12A.3.3 SOLID MASONRY

Solid masonry shall be solid concrete or clay masonry units laid contiguously in mortar.

The bonding of adjacent wythes in bearing and nonbearing walls shall conform to one of the following methods:

- HEADERS. The facing and backing shall be bonded so that not less than 4 percent of the exposed face area is composed of solid headers extending not less than 3 inches into the backing. The distance between adjacent full length headers shall not exceed 24 inches vertically or horizontally. Where backing consists of two or more wythes, the headers shall extend not less than 3 inches into the most-distant wythe or the backing wythes shall be bonded together with separate headers whose area and spacing conform to this Subsection.

- METAL TIES. The facing and backing shall be bonded with corrosion-resistant unit metal ties or cross wires or approved joint reinforcement conforming to the requirements of Sec. 12A.3.4 for cavity walls. Units ties shall be of sufficient length to engage all wythes, with ends embedded not less than one inch in mortar, or shall consist of two lengths, the inner embedded ends of which are hooked and lapped not less than 2 inches.

Where the space between metal tied wythes is solidly filled with mortar the allowable stresses and other provisions for masonry bonded walls shall apply. Where the space is not filled, metal tied walls shall conform to the allowable stress, lateral support, thickness (excluding cavity), height, and mortar requirements for cavity walls.

12A.3.4 CAVITY WALL MASONRY

Cavity wall masonry is that type of construction made with brick, structural clay tile or concrete masonry units, or any combination of such units in which facing and backing are completely separated except for the metal ties which serve as bonding.

In cavity walls neither the facing nor the backing shall be less than 4 inches in thickness and the cavity shall not be less than 1 inch net in width nor more than 4 inches in width. The backing shall be at least as thick as the facing.

EXCEPTION:
Where both the facing and backing are constructed with solid units, the facing and backing may each be 3 inches in thickness.

The facing and backing of cavity walls shall be bonded with 3/16 inch diameter steel rods or metal ties of equivalent strength and stiffness embedded in the horizontal joints. There shall be one metal tie for not more than 4.5 square feet of wall area for cavity widths up to 3.5 inches. Where the cavity exceeds 3.5 inches net in width, there shall be one metal tie for not more than 3 square feet of wall area. Ties in alternate courses shall be staggered and the maximum vertical distance shall not exceed 36 inches. Rods bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical; in other walls the ends of ties shall be bent to 90 degree angles to provide hooks not less than 2 inches long. Additional bonding ties shall be provided at all openings, spaced not more than 3 feet apart around the perimeter and within 12 inches of the opening. Ties shall be of corrosion-resistant metal, or shall be coated with a corrosion-resisting metal or other approved protective coating.

12A.3.5 GROUTED MASONRY - MULTIWYTHE WALLS

Grouted masonry is that form of construction made with brick or solid concrete units in which interior joints of masonry are filled by pouring grout therein as the work progresses. Only Type M or Type S mortar shall be used. When reinforced in accordance with subsection (C) below masonry shall be classified as reinforced grouted masonry.

Grouting procedures for the space between wythes shall conform to the requirements given below. Coarse grout may be used in grout spaces 2 inches or more in width. Coarse grout shall be used where the least dimension of the grout space exceeds 5 inches.

(A) LOW LIFT. Low lift grouted construction procedures are as follows:

Masonry headers shall not project into the grout space.

1. All longitudinal vertical joints shall be grouted and shall not be less than 3/4 inch in thickness for unreinforced construction and 1-1/2 inches in width for reinforced construction, but not less than that required to maintain grout thicknesses between masonry units and reinforcement. In members of three or more wythes in thickness, interior bricks shall be embedded into the grout so that at least 3/4 inch of grout surrounds the side and ends of each unit. Floaters may be used where the grout space exceeds 5 inches in width. All grout shall be puddled immediately after pouring.

2. One exterior wythe may be carried up 18 inches before grouting, but the other exterior wythe shall be laid up and grouted in lifts not to exceed six times the width of the grout space with a maximum of 8 inches.

3. If the work is stopped for one hour or longer, the horizontal construction joints shall be formed by stopping all wythes at the same elevation and with the grout 1 inch below the top.

(B) HIGH LIFT. High lift grouted construction procedures are as follows:

1. The two wythes shall be bonded together with wall ties. Ties shall be not less than No. 9 wire in the form of rectangles 4 inches wide and 2 inches in length less than the overall wall thickness. Kinks, water, drips, or deformations shall not be permitted in the ties. Approved equivalent ties may also be used. One wythe of the wall shall be built up not more than 18 inches ahead of the other tier. Ties shall be laid not to exceed 24 inches on center horizontally and 16 inches on center vertically for running bond and not more than 24 inches on center horizontally and 12 inches on center vertically for stacked bond.

2. Cleanouts shall be provided for each pour by leaving out every other unit in the bottom tier of the section being poured, or by cleanout openings in the foundation. During the work, mortar fins and any other foreign matter shall be removed from the grout space by means of a high pressure jet stream of water, air jets, or other approved procedures. Material falling to the bottom of the grout space shall be thoroughly removed. The cleanouts shall be sealed after inspection and before grouting.

3. The grout space (longitudinal vertical joint) shall not be less than 3 inches in width nor less than the thickness required by the placement of steel with the required clearances and shall be poured solidly with grout.

EXCEPTION:
If the grout space contains no horizontal steel, it shall be at least 2 inches.

4. Vertical grout barriers or dams shall be built of solid masonry across the grout space the entire height of the wall to control the flow of the grout horizontally. Grout barriers shall be

not more than 30 feet apart. Reinforcement, if it is present, shall be continuous through the barrier. In work that is part of the seismic resisting system, the grout barriers shall be constructed so as to form keys, at least 3/4 inch deep, with the grout except that construction providing equivalent irregular surfaces may be used where appropriate.

5. Grout shall be a plastic mix suitable for pumping without segregation of the constituents, and shall be mixed thoroughly. Grout shall be placed by pumping or by an approved alternate method and shall be placed before any initial set occurs.

6. Grouting shall be done in a continuous pour, in lifts not exceeding 6 feet. The full height of each lift shall be consolidated by mechanical vibrating during placing and reconsolidated after excess moisture has been absorbed, but before plasticity is lost. The grouting of any section of a wall between control barriers shall be completed in one day with no interruptions greater than one hour.

7. Inspection during grouting shall be provided in accordance with Sec. 12A.7.

(C) REINFORCED CONSTRUCTION. All required reinforcement except masonry joint reinforcement and column ties conforming to the paragraph below shall be embedded in grout. All other reinforcement shall be embedded in mortar or grout. All vertical reinforcement shall be held firmly in place during grouting by a frame or suitable equivalent devices. All horizontal reinforcement in the grout space shall be tied to the vertical reinforcement or held in place during grouting by equivalent means.

The thickness of mortar between masonry units and reinforcement shall not be less than 1/4 inch, except that where allowed 1/4 inch bars or less may be laid in horizontal mortar joints at least twice the thickness of the wire diameter. See Sec. 12A.6.3(F) and Chapter 12.

The thickness of grout between masonry units and reinforcement shall not be less than 1/4 inch where fine grout is used nor 1/2 inch where coarse grout is used. See Sec. 12A.1.17 and 12A.2.4.

See Chapter 12 for stacked bond limitations.

12A.3.6 HOLLOW UNIT MASONRY

Hollow unit masonry is that form of construction made with hollow masonry units made from concrete, burned clay, or shale.

Where two or more hollow units are used to make up the thickness of any unreinforced wall, the stretcher course shall be bonded at vertical intervals not exceeding 34 inches by lapping at least 4 inches over the unit below or by lapping at vertical intervals not exceeding 17 inches with units which are at least 50 percent greater in thickness than the units below; or by bonding with corrosion-resistant metal ties conforming to the requirements for cavity walls. There shall be one metal tie for not more than each 4.5 square feet of wall area. Ties in alternate courses shall be staggered, and the maximum vertical distance between ties shall not exceed 18 inches, and the maximum horizontal distance shall not exceed 36 inches. Walls bonded with metal ties shall conform to the requirements for allowable stress, lateral support, thickness (excluding cavity), height, and mortar for cavity walls.

Hollow unit masonry construction, where certain cells are continuously filled with concrete or grout, and reinforcement, in accordance with Subsection (A) below, is embedded therein shall be classified as reinforced hollow unit masonry. Reinforced hollow unit masonry shall generally be one wythe in thickness. If constructed of more than one wythe, each wythe shall be designed as a separate element or wall or the wythes shall be bonded together by means approved by the Regulatory Agency. This bonding shall be designed so the wythes shall act as a unit.

(A) GROUTING PROCEDURES FOR REINFORCED CONSTRUCTION. Units shall be laid with mortar in accordance with Sec. 12A.2.1. Only Types M or S mortar shall be used. Where only certain vertical cells are to be filled, the walls and cross webs of these cells shall be full bedded in mortar to prevent grout leakage. Vertical cells to be filled shall have vertical alignment sufficient to maintain a clear, unobstructed continous vertical cell measuring not less than 2 inches by 3 inches. If walls are battered or if alignment is offset, the 2 inch by 3 inch clear opening shall be maintained as measured from course to course.

12A.3.6 Cont.

Overhanging mortar fins projecting into the grout space shall be removed.

Coarse grout may be used in hollow masonry units having an area of 10 square inches with a least dimension of 3 inches. Coarse grout shall be used when the least diamension of the grout space exceeds 5 inches and where otherwise required.

Except as provided in Chapter 12, all reinforcing except ties and masonry joint reinforcement, where permitted, shall be embedded in grout. Longitudinal horizontal reinforcing shall be placed in bond beams, except as permitted for masonry joint reinforcement.

Vertical reinforcement shall be positively held in position at top and bottom and at intervals not exceeding 192 diameters of the reinforcement.

The thickness of the grout between the masonry units and reinforcing shall be a minimum 1/4 inch for fine grout and 1/2 inch for coarse grout. See Sec. 12A.1.17 and 12A.2.4. See Chapter 12 for stacked bond limitations.

Grouting procedures shall conform to the requirements given below. When grouting is stopped for one hour or longer, horizontal construction joints shall be formed by stopping the pour of grout at least 1/2 inch above or below a bed joint.

1. Low Lift. Low lift grouted construction procedures are as follows:

a. Hollow units shall be laid to a height not to exceed 4 feet 8 inches prior to filling cells with grout; grouting shall not be in lifts greater than 4 feet.

b. All cells containing reinforcement shall be filled solidly with grout. All grout shall be consolidated at the time of pouring by puddling or vibrating. When the grout lift exceeds 2 feet, the grout shall be reconsolidated after excess moisture has been absorbed, but before workability is lost.

c. Reinforcing shall be in place prior to grouting.

2. High lift. High lift grouted construction procedures are as follows:

a. Units may be laid up to 8 inches higher than the total height of the grout lift which shall not exceed 16 feet for walls 8 inches or more in nominal thickness nor 8 feet for thinner walls.

b. Cleanouts shall be provided in the foundation or by omitting face shells in the bottom course of each cell to be grouted to facilitate cleanout which shall be accomplished by means of a high pressure jet stream of water, air jets, or other approved procedures. Material falling to the bottom of the grout space and other debris shall be thoroughly removed.

c. The cleanouts shall be sealed after inspection and before grouting. Grout shall be a workable mix suitable for pumping without segregation of the constitutents and shall be mixed thoroughly. Grout shall be placed by pumping or by an approved alternate method and shall be placed before initial set or hardening occurs.

d. Grouting shall be done in a continuous pour and may be done in partial lifts.

The full height of each lift shall be consolidated by mechanical vibrating during placing, and reconsolidated after excess moisture has been absorbed but before workability is lost. The grouting of any section of a wall shall be completed in one day with no interruptions greater than 1.5 hours.

e. Inspection during grouting shall be provided in accordance with Sec. 12A.7.

12A.3.7 GLASS MASONRY

Masonry of glass blocks may be used in non-loadbearing exterior or interior walls and in openings, either isolated or in continuous bands, provided the glass block panels have a minimum thickness of 3 inches at the mortar joint.

12A.3.7 Cont.

The panels shall be supported laterally to resist the horizontal forces specified in Chapter 8. Glass block panels for exterior walls shall not exceed 144 square feet of unsupported wall surface nor 15 feet in any dimension. For interior walls, glass block panels shall not exceed 250 square feet of unsupported area nor 25 feet in any dimension.

Glass block shall be laid in Type N mortar by proportion. Both vertical and horizontal mortar joints shall be at least 1/4 inch and not more than 3/8 inch thick and shall be completely filled.

Every exterior glass block panel shall be provided with 1/2 inch expansion joints at the sides and top. Expansion joints shall be entirely free of mortar, and shall be filled with resilient material.

Sec. 12A.4 DETAILED REQUIREMENTS

Masonry shall be designed to resist all vertical and horizontal load effects including effects of eccentricity of application of vertical loads. Unreinforced masonry shall not be loaded in direct tension. Structural and nonstructural elements including partitions shall be designed for seismic forces induced by their own weight. Design of structural masonry that is not part of the seismic system shall consider the effects of seismic drift in accordance with Sec. 3.8.

Except where specifically allowed otherwise, stresses shall be calculated on actual net dimensions of masonry considering reductions for raking, tooling, and other joint treatments and partial bed or head joints where applicable. Where required by the Regulatory Agency, Chapter 12, and this Chapter, or by other governing provisions, inspections and tests shall be provided. In addition where called for or where required by the use of design stresses so specifying, Special Inspection shall be provided.

12.A.4.1 COMBINATION OF DISSIMILAR UNITS OR CONSTRUCTION

In walls or other structural members composed of different kinds or grades of units, materials, mortars, or construction types, the maximum stress shall not exceed the allowable stress for the weakest of the combination of units, materials, mortars, or construction types of which the member is composed. Alternatively, provided the effects of different modulii of elasticity are accounted for in design, the maximum stress shall not exceed the allowable stress for the material occurring at the point of stress consideration. The net thickness of any facing unit which is used to resist stress shall not be less than 1.5 inches.

In cavity walls composed of different kinds or grades of units or mortars the maximum stress shall not exceed the allowable stress for the weaker of the combination of units and mortars where both wythes are loadbearing; where only one wythe is loadbearing maximum stresses shall not exceed the allowable stresses for the units and mortars of that wythe.

12A.4.2 THICKNESS OF WALLS

All masonry walls shall be designed so that allowable stresses are not exceeded and that the provisions of Table 12A-1 are satisfied:

EXCEPTION:
The maximum thickness ratio of Table 12A-1 may be increased when justified by substantiating data.

12A.4.3 CHASES AND RECESSES

Chases and recesses in masonry walls shall be designed and constructed so as to satisfy the required strength or fire resistance of the wall. See Sec. 12A.2.8.

12A.4.4 HOLES, PIPES, AND CONDUITS

Pipes, conduits, and similar items may be sleeved through masonry with sleeves large enough to pass hubs and couplings. Pipes, conduits, and similar items may be embedded in masonry, provided all applicable provisions for Sec. 6.3 of ACI Standard 318 are satisfied. The design shall consider the net section at the location of the weakening element. Details shall be shown on the approved plans. In applying ACI Standard 318, the terms "concrete" and "structural concrete" shall mean masonry. (See Sec. 12A.2.8.) Unless all of the above requirements are satisfied, holes and embedments are not allowed.

12A.4.5 ARCHES AND LINTELS

Members supporting the vertical load of masonry shall be of noncombustible materials.

12A.4.6 ANCHORAGE

Masonry walls that meet or intersect shall be bonded or anchored as required by Sec. 12A.2.1 and 12A.2.2 except where separation is provided for in the design. Masonry walls shall be anchored to the roof and floors as required by Sec. 3.7.6. Structural members framing into or supported on walls or columns shall be bonded or anchored thereto.

12A.4.7 END SUPPORT

Beams, girders, or other similar concentrated loads supported by a wall or column shall have a bearing at least 3 inches in length upon solid or grouted elements of masonry not less than 4 inches thick or upon a metal bearing plate of adequate design and dimensions. The loads shall be distributed to the wall or column or to a continuous reinforced masonry member or by other approved means.

Joists, precast planks, and similar elements shall have a bearing at least 2.5 inches in length upon solid or grouted masonry elements or other provisions shall be made to distribute the loads to the masonry.

Anchorage to the masonry shall conform to Chapter 3.

12A.4.8 DISTRIBUTION OF CONCENTRATED LOADS

In calculating wall stresses concentrated loads may be distributed over a maximum length of wall not exceeding the center-to-center distance between loads.

Where the concentrated loads are not distributed through a structural element the length of wall considered shall not exceed the width of the bearing plus four times the wall thickness.

Concentrated loads shall not be considered to be distributed across continuous vertical joints unless reinforced horizontal elements designed to distribute the concentrated loads are employed.

Sec. 12A.5 <u>STRENGTHS AND ALLOWABLE STRESSES</u>

Material strength determinations and allowable stresses shall conform to the requirements of this Section.

12A.5.1 MASONRY

Except for the stresses listed in Table 12A-2 which are applicable to unreinforced masonry, the design of masonry is based on the compressive strength f'_m. The strength f'_m is reduced when the design is based on the alternate design procedure for unreinforced brick masonry of Sec. 12A.6.2. The higher stresses allowed in the Tables in this Chapter under the heading "Special Inspection Required" may only be used when all the applicable requirements for Special Inspection have been met; see Sec. 1.6 and 12A.7.

(A) DETERMINATION OF MASONRY COMPRESSIVE STRENGTH f'_m. When required for design, the value of f'_m shall be determined by tests of masonry assemblies in accordance with 12A.5.1(A).1 or shall be assumed in accordance with 12A.5.1(A).2.

1. Determination of f'_m by Prism Tests. When the masonry strength is to be established by tests, the procedures shall conform to the provisions of Sec. 12A.6 with tests made both prior to and during construction.

2. Assumed Compressive Strength f'_m. When prism tests are not made as in 12A.5.1(A).1, f'_m may be assumed as listed in Table 12-3 provided other tests are made and certifications are furnished when required by the footnotes to Table 12-3 or by the provisions upon which the design is based.

The tests in 12A.5.1(A).1 and 2 shall not qualify the masonry for the stresses entitled "Special Instruction" unless Special Inspection fully conforming to Sec. 1.6 and 12A.7 is provided.

12A.5.1 Cont.

(B) ALLOWABLE STRESSES FOR MASONRY. Except for unreinforced masonry designed under the provisions of Sec. 12A.6.2, the allowable stresses for unreinforced masonry are given in Table 12A-2 and for reinforced masonry the allowable stresses are given in Table 12A-4. See also Sec. 12A.6.3(E).

If used for design, the value of f'_m shall be clearly shown on the plans.

12A.5.2 STEEL

Stresses in reinforcement shall not exceed the following:

	POUNDS PER SQUARE INCH
TENSILE STRESS:	
For deformed bars with a yield of 60,000 pounds per square inch or more and in sizes No. 11 and smaller	24,000
Joint reinforcement, 50 percent of the minimum specified yield point for the particular kind and grade of steel used, but in no case to exceed	30,000
For all other reinforcement	20,000
COMPRESSIVE STRESS IN COLUMN VERTICALS:	
40 Percent of the minimum yield strength, but not to exceed	24,000
COMPRESSIVE STRESS IN FLEXURAL MEMBERS	
For compressive reinforcement in flexural members, the allowable stress shall not be taken as greater than the allowable tensile stress shown above	
The modulus of elasticity of steel reinforcement may be taken as	29,000,000 to 30,000,000

12A.5.3 BOLTS

The allowable shear loads on bolts shall not exceed the values given in Table 12A-5. See Sec. 12A.2.7 for construction requirements.

Sec. 12A.6 <u>DESIGN REQUIREMENTS</u>

The design of masonry elements shall conform to the appropriate provisions of this Section. The higher stresses allowed in the Tables in this Chapter under the heading "Special Inspection Required" may only be used when all of the requirements for Special Inspection have been met; see Sec. 1.6 and 12A.7. The load combinations of Sec. 3.7 shall be investigated. All plans shall clearly show the specified value of f'_m used in design. All stresses and capacities shall be based on actual net dimensions, thickness, and sections.

12A.6.1 DESIGN PROCEDURE FOR UNREINFORCED MASONRY

The design of unreinforced masonry shall be based upon a rational analysis using accepted engineering practice and linear stress and strain relationships. Alternate procedures for design are given in Sec. 12A.6.2.

(A) LIMITATIONS. The stresses on masonry elements including the stresses at the extreme fibers of the masonry element resulting from the combined effects of flexural and axial loads shall not exceed those given in Table 12A-2. The allowable compressive stresses of Table 12A-3 are applicable only if the thickness ratios of Table 12A-2 are not exceeded.

The allowable stresses for compression of Table 12A-3 shall be reduced by 20 percent when applied to columns.

Each wythe of cavity walls shall be designed separately for the loadings and effects imposed on it. The wythes shall not be assumed to act compositely.

(B) EFFECTIVE THICKNESS. For solid walls and metal tied walls, the effective thickness shall be determined as for cavity walls unless the collar joints in such walls are filled with mortar or grout.

For cavity walls loaded on both wythes, each wythe shall be considered to act independently and the effective thickness of each wythe shall be taken as its actual thickness. For cavity walls loaded on one wythe only, the effective thickness shall be taken as the actual thickness of the loaded wythe.

For rectangular columns, the effective thickness shall be taken as its actual thickness in the direction considered. For nonrectangular columns, the effective thickness shall be taken as equal to 3.5 times its radius of gyration r about the axis considered.

Where raked or similar mortar joints are used, the thickness and length of the member shall be reduced for stress consideration in accordance with the depth of the raking.

(C) ECCENTRICITY NORMAL TO AXES OF MEMBER. In solid walls and columns, the eccentricity of the load shall be considered with respect to the centroidal axis of the member.

In cavity walls loaded on one wythe, the eccentricity shall be considered with respect to the centroidal axis of the loaded wythe. In cavity walls loaded on both wythes, the load shall be distributed to each wythe according to the eccentricity of the load about the centroidal axis of the wall.

For members composed of different kinds or grades of units or mortar, the variation in the moduli of elasticity shall be taken into account and the eccentricity shall be considered with respect to the center of resistance or the centroidal axis of the transformed area of the member.

(D) EFFECTIVE HEIGHT. Where a wall is laterally supported top and bottom, its effective height shall be taken as the actual height of the wall. Where there is no lateral support at the top of the wall, its effective height shall be taken as twice the height of the wall above the bottom lateral support.

Where a column is provided with lateral supports in the directions of both principal axes at both top and bottom, the effective height in any direction shall be taken as the actual height. The actual height shall be taken as not less than the clear distance between the floor surface and the underside of the deeper beam framing into the column in each direction at the next higher floor level.

Where a column is provided with lateral support in the directions of both principal axes at the bottom and in the direction of one principal axis at the top, its effective height relative to the direction of the top support shall be taken as the height between supports and its effective height at right angles to this shall be taken as twice its height above the lower support.

In the absence of lateral support at the top, the effective height of a column relative to both principal axes shall be taken as twice its height abouve the lower support.

(E) CROSS-SECTIONAL AREA. For solid walls and columns, A_g shall be taken as the actual gross cross-sectional area of the member. For metal-tied walls, A_g shall be determined as for cavity walls unless the collar joints in such walls are filled with mortar or grout.

For cavity walls loaded on one wythe, A_g shall be taken as the actual gross cross-sectional areas of the loaded wythe.

In hollow unit construction, stresses shall be based on net areas.

Where raked or similar mortar joints are used, the thickness used in determining A_g or net areas shall be reduced accordingly.

(F) STIFFNESS. When used for design, the moduli of elasticity or rigidity may be assumed from values that would be applicable to similar masonry construction designed under other provisions of this Chapter. When the stiffness cannot or is not determined in this manner, supporting data shall be submitted.

(G) SHEAR WALLS. Design of shear walls shall conform to the applicable provisions of Sec. 12A.6.4.

(H) LOADS PERPENDICULAR TO CAVITY WALLS. The distribution to each cavity wall wythe of loads perpendicular to the plane of the wall shall consider relative wythe flexural rigidities, wythe end support conditions, and continuity or lack of continuity of each wythe.

12A.6.2 ALTERNATE DESIGN PROCEDURES FOR UNREINFORCED MASONRY

Unreinforced brick masonry using solid clay units and unreinforced concrete masonry may be designed by the alternate provisions following. The requirements of Sec. 12A.6.1 shall apply except as specifically modified.

(A) UNREINFORCED BRICK MASONRY USING SOLID CLAY UNITS. Unreinforced brick masonry using solid clay units may be designed under the applicable cited provisions of the "Building Code Requirements for Engineered Brick Masonry," Brick Institute of America, 1969 (B1A-1969) subject to the design and construction limitations listed.

1. Design shall conform to B1A-1969 Sec. 4.7.1 through 4.7.12 excluding Sec. 4.7.9, 4.7.10, and 4.7.12.5.

2. Materials shall conform to B1A-1969 Sec. 2.2.1 and 2.2.2.1.

3. Mortar joints shall conform to B1A-1969 Sec. 5.2.1

4. Construction shall be solid masonry, cavity wall or grouted masonry - multiple wythe.

5. Allowable stresses shall conform to B1A-1969 Table 3 with the following modifications:

a. The words "without inspection" of B1A-1969 Table 3 shall mean "without special inspection." The words "with inspection" shall mean "with special inspection."

b. Allowable compressive and bearing stresses without special inspection shall be 1/2 of those with special inspection.

c. Allowable flexural tension stresses without special inspection shall be 1/2 of those with special inspection.

d. Allowable shear stresses without special inspection shall be 1/2 of those with special inspection.

6. Moduli of Elasticity shall conform to Table 12A-5, this Chapter.

7. Brick masonry ultimate compressive strength f'_m for use in this alternate procedure shall be taken from Table 12A-4, or shall be 82 percent of the values obtained by prism testing according to Section 12A.8 ($h/t = 2$).

8. References to B1A-1969 Sec. 4.7.9 and 4.7.10 shall mean reinforced masonry conforming to the provisions for some of this chapter.

9. Footnote 4 to Table 12A-3 is applicable.

10. The Slenderness requirements of Sec. 12A.4.2 this Chapter shall be satisfied, however, these requirements and the slenderness limits of the alternate procedure may be waived in accordance with Sec. 1.5 of Chap. 1. Particular care shall be paid to requirements for stress and stability under reduced vertical loads.

11. For walls and elements subject to bending in one direction only, where the ratio e/t exceeds 1/3, the maximum tension and flexural compression stresses, assuming linear stress distribution, shall not be exceeded.

12. Design of shear walls shall comply with all applicable provisions of this chapter. Loading combinations shall include reduced vertical loads in combination with seismic loads, where applicable. The allowable shear stress increase shall consider this vertical load reduction.

(B) UNREINFORCED CONCRETE MASONRY. Unreinforced concrete masonry using solid or hollow units and grouted or ungrouted construction, may be designed using the applicable cited supplemental provisions of the "Specification for the Design and Construction of Load-bearing Concrete Masonry," National Concrete Masonry Assoc., 1979 (NCMA - 1979) subject to the design and construction limitations listed.

1. Design shall conform to NCMA - 1979 Sections. 3.3.1, 3.3.2, and 3.8.6 through 3.8.8 except that allowable stresses and resistances therein are for work only with special inspection; for work without special inspection they shall be reduced by 50 percent.

2. Mortal shall conform to NCMA - 1979 Sec. 2.2.2.2.

3. Joints shall conform to NCMA - 1979 Sec. 4.2.3.2.

4. BEARING STRESS (f_{br})

On full area, $F_{br} = .25\, f'_m$

On 1/3 area or less, $F_{br} = .30\, f'_m$

This increase shall be permitted only when the least distance between the edges of the loaded and unloaded area is a minimum of 1/4 of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonable concentric area greater than 1/3, but less than the full area shall be interpolated between the values given.

(C) DESIGN, UNREINFORCED HOLLOW CLAY MASONRY.

GENERAL. Unreinforced masonry using hollow clay units may be used when designed in accordance with the provisions of this Section. The allowable stresses shown herein are for work only with special inspection, for work without special inspection these allowable stresses shall be reduced by 50 percent.

1. COMPRESSION IN WALLS AND COLUMNS

(a) AXIAL LOADS. Streses due to compressive forces applied at the centroid of the member may be computed assuming uniform distribution over the effective area. The allowable axial compressive stress is given by:

$$F_a = 0.20\, f'_m\, [1-(h'/40t)^3] \text{ walls} \qquad \text{Eq. 12A-1}$$

$$F_a = 0.18\, f'_m\, [1-(h/30t)^3] \text{ columns} \qquad \text{Eq. 12A-2}$$

in which

f_m' = ultimate compressive strength of masonry. For assumed values of f'_m use Table 12A-1.

h = clear distance in inches, between supporting or stiffening elements (vertical or horizontal). Effective height or length different from clear distance may be used if justified.

t = effective thickness (the minimum effective thickness in the case of columns).

ASSUMED VALUES OF f'_m for use in Eqs. 12A-1 and 12A-2.

The design ultimate compressive stress of masonry, f'_m may be assumed based upon the compressive strength of the units and mortar to be used. Values of f'_m which may be assumed are presented in Table 12A-1.

(b) BEARING STRESS (f_{br})

On full area, $F_{br} = .25 f'_m$ Eq. 12A-3

On 1/3 area or less, $F_{br} = .30 f'_m$ Eq. 12A-4

This increase shall be permitted only when the least distance between the edges of the loaded and unloaded area is a minimum of 1/4 of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than 1/3, but less thean the full area shall be interpolated between the values given.

2. BENDING OR COMBINED BENDING AND AXIAL LOADS

Stresses due to combined bending and centroidally applied axial load shall satisfy the requirements of Section 12A.6.3(b) where F_a is given by Equation 12A-1 and 12A-2.

3. FLEXURAL DESIGN

(a) Tensile stresses due to flexure shall not exceed the values given in Section 12A.6.2(C)3b where:

$F_b = Mc/I$ Eq. 12A-5

and:

f_b = computed flexural stress due to bending loads only.

M = design moment on a section.

c = distance from neutral axis to extreme fiber.

I = moment of inertia of the section considered.

(b) TENSILE STRESS - FLEXURAL (F_t). With no tensile reinforcement in masonry values for tension normal to head joints are for running bond; no tension is allowed across head joints in stack bond masonry.

Tension Normal to Bed Joints (net bedded area)

	Clay Units
Hollow Units, F_t =	24 psi
Tension Normal to Head Joints	
Hollow Units, F_t =	48 psi

Stresses are calculated on net bedded area.

Compression stresses due to flexure (F_b) shall not exceed 0.33 f'_m.

4. SHEAR IN FLEXURAL MEMBERS AND SHEAR WALLS

(a) SHEAR IN FLEXURAL MEMBERS

$v_m = V/A_e$ Eq. 12A-6

where:

v_m = design shear stress with no shear reinforcement. The allowable shear stress, v_m, may be equal to $1.0\sqrt{f'_m}$ but not to exceed 50 psi.

V = total design force.

A_e = effective area.

Where v_m as computed by the foregoing equation exceeds the allowable shear stress, V_m, web reinforcement shall be provided and designed to carry the total shear force in accordance with the requirements of reinforced masonry in Section 12.6.3(c).

(b) SHEAR WALLS WITH NO SHEAR REINFORCEMENT SHALL BE DESIGNED USING THE FOLLOWING EQUATIONS:

No shear reinforcment

$$a/L < v_m = 1/3 \left[4 - \frac{a}{L}\right] f'_m, \text{ 50 max.} \qquad \text{Eq. 12A-7}$$

$$A/L \geq 1, \; v_m = 1.0 \sqrt{f'_m}, \text{ 35 max.} \qquad \text{Eq. 12A-8}$$

where:

a = height of wall or segment for cantilevered condition, 1/2 height of wall or segment for fixed conditions top and bottom.

L = length of wall or segment.

The allowable shear stress in masonry may be increased by 0.2 f_{md}, where f_{md} is the compressive stress in masonry due to dead load only.

(c) SHEAR WALL OVERTURNING. Not more than 2/3 of the dead load shall be used to resist overturning due to horizontal forces. Any resultant tensile stresses shall be resisted by reinforcing in accordance with the requirements of Section 12A.6.3.

5. <u>CORBELS</u>

The slope of corbeling (angle measured from the horizontal to the face of the corbeled surface) shall not be less than 60 degrees. The maximum horizontal projection of corbeling from the plane of the wall shall not exceed 1/2 the wythe thickness for cavity walls or 1/2 the wall thickness for other walls.

12A6.3 DESIGN PROCEDURE FOR REINFORCED MASONRY

The design of reinforced masonry shall comply with this Section and be based on accepted engineering practice forthe "working stress" theory which incorporates the following principal assumptions:

- A section that is plane before bending remains panel after bending.
- Moduli of elasticity of the masonry and of the reinforcement remain constant.
- Tensile forces are resisted only by the tensile reinforcement.
- Reinforcement is completely surrounded by and bonded to masonry material so that they will work together as a homogeneous material within the range of working stresses.

Stresses shall not exceed those given in Sec. 12A.5 and this Section.

(A) FLEXURAL COMPUTATIONS. All members shall be designed to resist at all sections the maximum bending moment and shears produced by dead load, live load, and other forces as determined by the principles of continuity and relative rigidity. The clear distance between lateral supports of a beam shall not exceed 32 times the least width of the compressive flange or face.

In computing flexural stresses for masonry wall elements the effective length tributary to a reinforcing bar shall be limited to:

12A.6.3 Cont.

1. Running Bond. Six times the wall thickness.

2. Stacked Bond. Three times the wall thickness or the length of the masonry units for construction using stacked bond, whichever is less.

(B) COMBINED AXIAL AND FLEXURAL STRESSES. Members subject to combined axial and flexural streses shall be proportioned, except as modified by Chapter 12, so that the following formula is satisfied.

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{Eq. 12A-9}$$

where:

f_a = Computed axial unit stresses, determined from total axial load and effective area.

F_a = Axial unit stress permitted by this Chapter if member were carrying axial load only.

f_b = Computed flexural unit stress.

F_b = Flexural unit stress permitted based by this Chapter on elastic methods and design assumptions may be used.

Other interaction equations based on elastic methods and design assumptions may be used.

(C) SHEAR AND DIAGONAL TENSION. The shearing unit stress, v, in reinforced masonry flexural members shall be computed by:

$$v = \frac{V}{bjd} \qquad \text{Eq. 12A-10}$$

where:

b = The net effective width of a rectangular section or stem of I- or T-sections. The value of bd shall not exceed the net vertical shear area, net bedded area, nor the net cross-sectional area in hollow unit construction.

d = The effective depth.

j = Ratio of distance between centroid of compression and centroid of tension to depth, d. j may be assumed as 0.85 or j may be determined by a strain compatibility analysis.

In vertical joints of stacked bond construction, the vertical joints shall not be assumed to resist shearing stresses. Where the shear reinforcement is parallel to the vertical joints, reinforcement equal to the required shear reinforcement shall be provided perpendicular to the vertical joints at a spacing not to exceed 16 inches.

Where the values of the shearing unit stress computed by Formula 12A-10 exceeds the shearing unit stress, v_m, shear reinforcement shall be provided to carry the entire stress.

1. Shear Reinforcement. Shear reinforcement shall consist of:

a. Shear reinforcement bars perpendicular to longitudinal steel, or

b. Shear reinforcement bars anchored around or beyond the longitudinal steel and making an angle of 30 degree or more thereto, or

c. Longitudinal bars bend so that the axis of the inclined portion of the bar makes an angle of 15 degree or more with the axis of the longitudinal portion of the bar, or

d. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of masonry by the reinforcement.

Bars to be considered effective as shear reinforcement shall be anchored at both ends.

2. Required Area. The area of steel, A_v, required perpendicular to the longitudinal reinforcement shall be computed by the following formula:

$$A_v = \frac{Vs}{f_v jd} \qquad \text{Eq. 12A-11}$$

where:

V = the total shear, in pounds.

s = the spacing of bars in a direction parallel to that of the main reinforcement, inches.

f_v = the allowable unit stress in the shear reinforcement, psi.

Inclined bars shall be proportioned in accordance with the provisions of Paragraph 3 of this Subsection.

3. Bent Bars. Only the center 3/4 of the inclined portion of any longitudinal bar that is bent up for shear reinforcement shall be considered effective for that purpose, and such bars shall be bent around a pin having a diameter not less than 6 times the bar size.

When the shear reinforcement consists of a single bent bar or of a single group of parallel bars all bent at the same distance from the support, the required area, A_v, of such bars shall be computed by the following formula:

$$A_v = \frac{V}{f_v \sin a} \qquad \text{Eq. 12A-12}$$

where:

a = the angle between inclined web bars and the axis of the beam.

Where there is a series of parallel bars or groups bent up at different distances from the support, the required area shall be determined by the following formula:

$$A_v = \frac{Vs}{f_v jd\,(\sin a + \cos a)} \qquad \text{Eq. 12A-13}$$

4. Spacing of Shear Reinforcement. Where shear reinforcement is required it shall be so spaced that every 45 degree line extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of shear reinforcement.

(D) REINFORCEMENT DEVELOPMENT, ANCHORAGE, AND SPLICES. Reinforcement shall be arranged, placed, spliced, and anchored to develop design stresses therein and as specified in this Subsection and Chapter 12.

1. General Development Requirements. The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by embedment length or end anchorage or a combination thereof. For bars in tension, hooks may be used in developing the bars. Plain bars in tension shall terminate in standard hooks. Tension reinforcement may be anchored by bending and making it continuous with the reinforcement on the opposite face of the member, or anchoring it there.

The critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a minimum distance equal to the effective depth of the member or 12 bar diameters, whichever is greater, except at supports of simple spans and at the free end of cantilevers. Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where bent or terminated tension reinforcement is not longer rquired to resist flexure.

Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

a. Allowable shear stresses at the cutoff point do not exceed 2/3 of that permitted, or

b. Area of shear reinforcement in excess of that required is provided along each terminated bar over a distance from the termination point equal to 3/4 the effective depth of the member. The shear reinforcement shall be proportioned to provide 50 percent of the allowable shear capacity of the member based on the allowable shear stresses of Table 12A-4 for reinforcement taking no shear. The resulting spacing s shall not exceed $d/8\beta_b$ where β_b is the ratio of the area of bars cut off to the total area of bars at the section, or

c. The continuing bars provide double the area for flexure at the cutoff point and shear stresses do not exceed 3/4 of that permitted.

2. Positive Moment Reinforcement. At least 1/3 of the positive moment reinforcement in simple members and 1/4 the positive moment reinforcement in continuous members shall extend along the same face of the member into the support, and in beams at least 6 inches.

When a flexural member is part of the primary lateral load resisting system, the positive reinforcement required above to be extended into the support shall be anchored for its tension development length, l_d, or if the support is not of masonry construction, the reinforcement shall be anchored to develop its calculated stress at the face of the support.

At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that the required development length, l_d, determined in this Section does not exceed:

$$\frac{M_c}{V} + l_a \qquad \text{Eq. 12A-14}$$

Where M_c is the lesser moment capacity of the member, based on allowable stresses, determined from both the masonry and the reinforcement, and V is the maximum applied shear at the section. At the point of support, l_a shall be the sum of the embedment length supplied beyond the center of the support and the equivalent embedment length of any furnished hook or mechanical anchorage. At the point of inflection, l_a shall be limited to the effective depth of the member or $12d_b$, whichever is greater, where d_b is the diameter of the reinforcement.

3. Negative Moment Reinforcement. Tension reinforcement in a continuous, restrained, or cantilever member, or in any member of the primary lateral force resisting system, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

Negative moment reinforcement shall be developed into the span as required by Sec. 12A.6.3(D)1.

At least 1/3 of the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, $12d_b$, or 1/16 of the clear span, whichever is greater.

4. Special Members. Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment; such as: sloped, stepped, or tapered members; brackets; deep beams; or members in which the tension reinforcement is not parallel to the compression face.

5. Development Lengths. The basic development length, l_d, for deformed reinforcement shall be at least $0.0015\ d_b f_s$ but not less than 24 d_b for reinforcement of 40,000 psi yield strength nor 36 d_b for reinforcement over 40,000 psi yield strength, nor less than 12 inches for reinforcing bars and 6 inches for masonry joint reinforcement where:

d_b = the diameter of the smaller bar spliced, inches.

f_s = the specified bar calculated stress, psi.

Development lengths for plain reinforcing shall be twice that required for deformed reinforcement but not less than 12 inches.

EXCEPTIONS:
1. For deformed main compression reinforcement in columns that are not part of the seismic system, these values may be reduced to 18 d_b for bars of 40,000 psi yield strength and 27 d_b for bars over 40,000 psi yield strength.
2. In flexural members that are not part of the primary lateral load resisting system the development lengths may be reduced where excess reinforcement is provided. For these cases, the previously determined development lengths may be multiplied by the ratio of the area of reinforcement required by design to that provided.

6. Hooks. The term "hook" or "standard hook" as used herein shall mean:

a. A complete semicircular turn plus an extension of at least 4 bar diameters at the free end of the bar but not less than 2-1/2 inches, or

b. A 90 degree bend having a radius of not less than 4 bar diameters plus an extension of 12 bar diameters, or

c. For stirrup anchorage only, a 135 degree turn with a radius on the axis of the bar of 3 diameters, plus an extension of at least 6 bar diameters at the free end of the bar.

EXCEPTIONS:
1. The hook for ties placed in the horizontal bed joints, where permitted, shall consist of a 90 degree bend plus an extension of 32 bar diameters.
2. The hook for ties in the form of crossties as described in this Subsection may have a 90 degree hook at one end provided the 90 degree hooks are alternated with the 135 degree hooks along the bar.

Inside diameter bends shall be as required for concrete reinforcement. Hooks having a radius of bend of more than 6 bar diameters shall be considered merely as extensions to the bars. In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

For tension bars, the hook at its start (point of tangency) may be considered as developing not more than 3/8 of the allowable tensile stress or the development length, l_d, for reinforcement of 40,000 psi yield strength and not more than 5/16 of the allowable tensile stress of the development length for reinforcement over 40,000 psi yield strength.

Hooks shall not be considered effective in adding to the compressive resistance of bars.

Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

7. Splices. Splices shall be made only at such point and in such manner that the strength of the member will not be reduced. Splices shall be made by lapping the bars, by welding, or by mechanical connections. Lapped splices shall not be used for tension tie members.

Lengths of laps, in inches, for deformed reinforcement shall be at least 0.002 d_bF_s but not less than 40 d_b for reinforcement of 40,000 psi yield strength nor less than 60 d_b for reinforcement over 40,00 psi yield strength, nor less than 12 inches for reinforcing bars and 9 inches for masonry joint reincorcement. Lap lengths for plain reinforcing shall be twice that required for deformed bars but not less than 12 inches. The terms d_b and F_s shall be as defined in Section 12A6.3(D)5.

EXCEPTION:
For deformed main compression reinforcement in columns that are not part of the seismic system, the lap length may be reduced to 30 d_b for bars of 40,000 psi yield strength and 45 d_b for bars over 40,000 psi yield strength.

Welded or mechanical connections shall develop the yield strength of the bar in tension.

EXCEPTION:
For compression bars in columns that are not part of the seismic system and are not subject to flexure the compressive strength need only be developed.

8. Anchorage of Shear Reinforcement. Shear reinforcement shall be placed as close to the compression and tension surfaces of the member as cover requirements, practicability, and other steel will permit, and in any case the ends of single-leg, simple- or multiple-U stirrups shall be anchored by one of the following means.

a. A standard hook plus an effective embedment of 5/8 l_d for reinforcement of 40,000 psi yield strength or 11/16 l_d for reinforcement over 40,000 psi yield strength. The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member, d/2, and the start of the hook (point of tangency), or

b. Embedment above or below the mid-depth, d/2, of the compression side of members that are not part of the seismic system for a full development length l_d, or

c. Bending around the longitudinal reinforcement through at least 180 degrees. Hooking or bending stirrups around the longitudinal reinforcement shall be considered effective anchorage only when the stirrups make an angle of at least 45 degrees with deformed longitudinal bars not less in diameter than the stirrup bars.

Between the anchorage ends, each bend in the continuous portion of a transverse simple- or multiple-U-stirrup enclose a longitudinal bar, not less in diameter than the stirrup bars.

Longitudinal bars bent to act as shear reinforcement shall, in a region of tension, be continued with the longitudinal reinforcement and in a compression zone shall be anchored, above or below the mid-depth, d/2, as specified for development length in this Subsection.

Pairs of U stirrups or ties so placed as to form a closed unit shall be considered properly spliced when the laps satisfy the requirements of this Subsection.

9. Flexural Compression Reinforcement. Required flexural compression steel in members that are not part of the seismic system shall be anchored (enclosed) by ties or stirrups not less than 1/4 inch in diameter, spaced not further apart than 16 bar diameters or 48 tie diameters. Such ties or stirrups shall be used throughout the distance where compression steel is required.

Required flexural compression reinforcement in members that are part of the seismic system shall be anchored as required for column longitudinal reinforcement.

(E) REINFORCED MASONRY WALLS. Reinforced masonry bearing wall thicknesses shall conform to Sec. 12A.4.2 and to the requirements of this Subsection and Chapter 12.

1. Stresses. The axial stress in reinforcement masonry bearing walls shall not exceed the value determined by the following formula:

$$F_m = 0.20 f'_m [1 - (\frac{h}{40t})^3] \qquad \text{Eq. 12A-11}$$

where:

f_m = Compressive unit axial stress in masonry wall.

f_m' = Masonry compressive strength as determined by Sec. 12A.5.1. The value of f'_m shall not exceed 6000 psi.

t = Thickness of wall in inches.

h = Clear distance in inches, between supporting or stiffening elements (vertical or horizontal). Effective height or length different from distance may be used if justified.

2. Reinforcement. Reinforcement of walls and wall elements shall be provided for all loadings and other requirements of these Regulations. Except for more stringent requirements of Chapter 12 and this Chapter, as applicable, the minimum reinforcement ratio in each direction shall be .0007 and the sum of the ratios for each direction shall not be less than .002. Maximum reinforcement spacing shall not exceed 4 feet on center. Only horizontal reinforcement which is continuous in the wall shall be considered in computing the minimum area of reinforcement.

Where the reinforcement ratios are less than those in the preceding paragraph allowable stresses for unreinforced masonry shall be used. If reinforcement ratios are equal to or greater tahn these ratios, the stresses of Table 12A-4 may be used.

If the wall is constructed of more than two units in thickness, the minimum area of required reinforcement shall be equally divided into two layers, except where designed as retaining walls. Where reinforcement is added above the minimum requirements such additional reinforcement need not be so divided.

Horizontal reinforcement shall be provided at the top of footings, at the top of wall openings, at roof and floor levels and at the top of parapet walls. If continuous, these special bars may be considered in satisfying the minimum horizontal reinforcement ratios of Sec. 12.7.

There shall not be less than one No. 4 or two No. 3 bars on all sides of, and adjacent to, every opening which exceeds 24 inches in either direction, and such bars shall extend not less than the development length, but in no case less than 24 inches beyond the corners of the opening. The bars required by this paragraph shall be in addition to the minimum reinforcement required elsewhere.

3. Columns Constructed Within Walls. When the reinforcement in bearing walls is designed, placed, and anchored in position as for columns, the allowable stresses shall be as for columns. The length of the wall to be considered effective shall not exceed the center-to-center distance between concentrated loads nor shall it exceed the width of the bearing plus 4 times the wall thickness.

4. Shear Walls. Shear walls shall, additionally, comply with the provisions of Sec. 12A.6.4.

(F) REINFORCED MASONRY COLUMNS. The least dimension of every reinforced masonry column shall not be less than 12 inches.

EXCEPTION:
The minimum column dimension may be reduced to not less than 8 inches provided the design is based upon 1/2 the allowable stresses for axial load. Bending stresses need not be so reduced.

The axial load on columns shall not exceed:

$$P = A_g\,(0.18\,f'_m + 0.65\,\rho_g\,f_s)\left[1 - \left(\frac{h}{40}\right)^3\right] \qquad \text{Eq. 12-16}$$

where:

p = Maximum concentric column axial load.

A_g = The gross area of the columns with deductions for rakes and similar joint treatments.

f_m' = Compressive masonry strength as determined by Sec. 12A.5.1. The value f'_m shall not exceed 6000 psi.

12A.6.3 Cont.

ρ_g = Ratio of the effective cross-sectional area of vertical reinforcement to A_g.

f_s = Allowable stress in reinforcement; see Sec. 12A.5.2.

t = Least thickness of column in inches.

h = Effective height-clear distance in inches between supporting or stiffening elements. Effective height different from clear distance may be used if justified.

1. Vertical Reinforcement. The ratio ρ_g shall not be less than 0.5 percent nor more than 4 percent. The number of bars shall not be less than four, nor the size less than No. 3. Except as provided in Sec. 12A.2.4(D), the maximum bar size shall be No. 10. Splices shall conform to Sec. 12A.6.3(D)7.

2. Ties. All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars, as specified below, by the corner of a complete tie having an included angle of not more than 135 degrees or by a hook at the end of a tie. The corner longitudinal bars shall have lateral support provided by a complete tie enclosing the longitudinal bars.

Lateral ties shall be placed not less than 1.5 inches and not more than 5 inches from the surface of the column, and may be against the vertical bars or placed in the horizontal bed joints where permitted by Sec. 12A.3.5(C).

The spacing shall not be greater than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches.

Ties shall be at least No. 2 in size for No. 7 or smaller longitudinal bars and No. 3 size for No. 8 or larger longitudinal bars except that when No. 11 bars are allowed under the exception to Sec. 12A.2.4(D) the minimum tie size shall be No. 4.

EXCEPTION:
Ties placed in horizontal bed joints, where permitted by Sec. 12A.3.5(C) may be smaller in size than required above but not less than No. 2 in size, provided that the total cross-sectional area of such smaller ties crossing a vertical plane is equal to the area of the larger ties at their required spacing.

3. All column or longitudinal reinforcing shall be solidly embedded in grout.

12A.6.4 MASONRY SHEAR WALLS

The design of masonry shear walls and wall elements for in-plane shears shall conform to this Section and all applicable provisions of these Regulations. See Chapter 12 for stacked bond construction limitations based on construction categories.

(A) INTERSECTING WALLS AND MASONRY COLUMNS. Where shear walls intersect a wall or walls to form symmetrical T- or I-sections, the effective flange width shall not exceed 1/6 of the total wall height above the level being analyzed, and its overhanging width on either side of the shear wall shall not exceed six times the nominal thickness of the intersected wall for unreinforced masonry nor eight times the nominal thickness of the intersected wall for reinforced masonry.

Where shear walls intersect a wall or walls to form L or C sections, the effective overhanging flange width shall not exceed 1/16 of the total wall height above the level being analyzed nor six times the nominal thickness of the intersected for unreinforced masonry nor eight times the nominal thickness of the intersected wall for reinforced masonry.

Limits on effetive flange width may be waived when approved after a review of a written justification.

The vertical shear at the intersection of shear wall web and flange shall be considered in design.

12A.6.4 Cont.

(B) VERTICAL TENSION AND COMPRESSION STRESSES. Except as provided for masonry designed under the alternate design procedure of Sec. 12A.6.2(A) vertical stresses in shear walls shall be determined from the combined effects of vertical loads.

Allowable tension staresses for unreinforced masonry shall not be exceeded. Anchorage to the foundation shall be provided to resist calculated tension in unreinforced walls.

(C) HORIZONTAL ELEMENTS. Provisions shall be made for shear and flexural effects in horizontal elements of shear wall systems, such as beams that couple walls.

(D) WALL SHEAR. In computing the shear resistance of the wall, only the web shall be considered. For unreinforced masonry the depth of the web may be considered out to out of flanges.

Shear resistance of masonry shall be based on minimum net areas parallel to the shear. Both vertical and horizontal shear shall be considered. Continuous vertical and horizontal grout elements may be considered as part of the net area.

For reinforced masonry, the shear stress shall be computed by Formula 12A-10. Horizontal shear reinforcing, when required, shall be provided with that portion required to resist shear uniformly distributed and spaced out not more than 1/3 the wall depth as required by Sec. 12.7, whichever is less.

(E) BOUNDARY ELEMENTS. Boundary elements are members at the ends of shear walls which help resist overturning effects.

Unit compressive stresses in the masonry at wall openings shall conform to the requirements of this Chapter unless boundary elements conforming to the provisions of Sec. 12.7.2 are provided.

Reinforcement required to resist wall shear shall be terminated with a standard hook which terminates beyond the boundary reinforcing at the end of the wall sections. The hook may be turned up, down, or horizontally and shall be embedded in mortar or grout. Wall reinforcement terminating in boundary columns or beams shall be fully anchored into the boundary elements.

12A.6.5 SCREEN WALLS

Masonry units may be used in nonbearing decorative screen walls. Units may be laid up in panels with units on edge with the open pattern of the unit exposed in the completed wall.

The panels shall be capable of spanning between supports to resist horizontal forces. Wind loads shall be based on gross projected area of the panel.

(A) UNREINFORCED PANELS. Unreinforced panels are allowed only in Category A construction provided allowable stresses are not exceeded. Otherwise the panels shall be reinforced as provided in Sec. 12A.6.5(B).

(B) REINFORCED PANELS. All panels in Catagories B, C, or D construction shall be reinforced per Sec. 12A.4.1(D).

Sec. 12A.7 INSPECTIONS AND TESTS

Inspections shall be provided and tests shall be made in accordance with the requirements of this Section. The Regulatory Agency may for masonry work which it determines to be minor in nature waive requirements for certifications, inspections, tests, or some other items of Special Inspection.

Inspection shall be done to an extent that the Inspector(s) or testing agency can certify to the requirements of Sec. 1.6.4.

12A.7.1 FREQUENCY OF INSPECTIONS AND TESTS

For all masonry construction, Inspection, Certification, or Tests shall be provided when required by one or more of the following:

12A.7.1 Cont.

- When required by provisions of Chapter 12 and this Chapter.
- When in the opinion of the Regulatory Agency work involves unusual hazards.
- Where required by the approved Quality Assurance Plan or design documents.

Inspections, Certifications, or Tests may consist of one or more of those listed in Sec. 12A.7.2(A) and 12A.7.2(B), however, in order to qualify as Special Inspection all the applicable Certifications, Inspections, and Tests of Sec. 12A.7.2 shall be provided.

12A.7.2 SPECIAL INSPECTION AND TESTS

All applicable Special Inspections and Tests designated in Sec. 12A.7.2(A) and 12A.7.2(B) shall be provided when stresses entitled "Special Inspection" are used for design, when required by the items listed in Sec. 12A.7.1, and when Special Inspection is otherwise required.

(A) SPECIAL INSPECTION. Special Inspection shall be provided as follows:

- For the examination of materials and/or certification of materials for compliance.
- For the observation of measurement and mixing of field-mixed mortar and grout including checks on consistency.
- For the determination of the moisture conditions of the masonry units at the time of laying.
- For periodic observations of the laying of masonry units with special attention to joints including preparations prior to buttering, portions to be filled, shoving, etc.
- For observation of the bonding of units in the walls between wythes and at corners and intersections.
- For the proper placment of reinforcement including splices, clearances, and support.
- For observation of the construction of chases, recesses, and the placement of pipes, conduits, and other weakening elements.
- For inspection of grout spaces immediately prior to grouting including the removal of mortar fins as required, removal of dirt and debris, and the conditions at the bottom of the grout space. For high lift work this shall be done prior to the closing of cleanouts and shall also include the proper sealing of cleanouts.
- For the preparation, or supervision of preparation, of required samples such as mortar, grout, and prisms.
- For the observation of grout placement with special attention to procedures to obtain filling of required spaces, the avoidance of segregation, and proper consolidation and reconsolidation.

(B) TESTS AND/OR CERTIFICATIONS. Tests and/or certifications shall be performed and/or supplied as follows:

- For mortar, grout, and prisms. One prism test series shall be made for each 5000 square feet of wall. Alternatively, a series of both mortar and grout tests shall be made on the first three consecutive days of the work and on each third day thereafter.

  In addition, when f'_m is equal to or greater than 2600 psi or when f'_m is to be established by tests, a minimum of three prism test series shall be made during the progress of the work. When f'_m is to be established by tests there shall be an initial prism test series prior to the start of construction.

  The requirements for numbers of test series apply separately for each variation of type of masonry construction except for the total number for a building.

12A.7.2 Cont.

- For masonry units. When shipments of masonry units are not identified and accompanied by certification one series of tests for strength, absorption, saturation, moisture content, shrinkage, and modulus of rupture shall be made for each 5000 square feet of wall or equivalent. When the reference document or standard for the units has no acceptance or rejection limits for a test, the test need not be made.

- Grouted masonry, Seismic Performance Category D. One series of core tests for shear bond shall be made for each 5000 square feet of wall or equivalent.

- For cement used for mortar and grout, certification acceptable to the Regulatory Agency shall accompany the cement when the required volume of cement exceeds 500 sacks.

- For reinforcement. One tensile and bend test shall be made for each 2-1/2 tons or fraction thereof of each size of reinforcing. Testing is not required if the reinforcement is identified by heat number and is accompanied with a certified report of the mill analysis.

- For plant mix ("transit mix") grout a certificate conforming to both Section 14.1 and 14.2 of ASTM C-94 shall accompany the plant mix. Substitute "grout" for "concrete" in ASTM C-94. The requirements for the testing of grout shall also apply.

- For other tests performance shall be as indicated in the Approved Quality Assurance Plan.

  Where the number of tests or test series is not defined, one test or test series, as applicable, shall be made for each 5000 square feet of wall or equivalent.

12A.7.3 LOAD TESTS

When a load test is required the member or portion of the structure under consideration shall be subject to a superimposed load equal to twice the specified live load plus 1/2 of the dead load. This load shall be left in position for a period of 24 hours before removal. If, during the test or upon removal of the load, the member or portion of the structure shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. A flexural member shall be considered to have passed the test if the maximum deflection "D" at the end of the 24 hour period neither exceeds:

$$D = \frac{L}{200}$$

$$\text{nor } D = \frac{L^2}{4000\ t}$$

and the beams and slabs show a recovery of at least 75 percent of the observed deflection within 24 hours after removal of the load where:

L = Span of the member in feet.

t = Thickness or depth of the member in feet.

12A.7.4 REPORTING

Reporting and compliance procedures shall conform to Sec. 1.6.4.

Sec. 12A.8 TEST CRITERIA

Masonry prisms, mortar and grout samples, and masonry cores shall be prepared and tested in accordance with the procedures in this Section.

12A.8.1 MASONRY PRISMS

Requirements for prisms shall be those of ASTM E447, except as modified by this Section.

Prisms shall be built of the same materials, under the same conditions, and insofar as possible, with the same bonding arrangements as for the structure including the lapping of units except that for prisms which are one masonry unit in length, the units may be laid in stacked bond. The moisture content of the units at the time of laying, consistency of mortar, and workmanship shall be the same as will be used in the structure for each type of construction.

Prisms shall be not less than 12 inches high and shall have a height to minimum thickness dimension ratio of not less than 2.0 nor more than 5.0. Ungrouted hollow masonry unit prisms shall be not less than one masonry unit in length. Solid grouted prisms of hollow units shall have a minimum length of one complete cell with cross webs. Solid masonry unit prisms or solid filled prisms shall be not less than 4 inches in length. The thickness and type of construction of the specimen shall be representative of the masonry element under consideration.

Cores for hollow unit masonry shall not be filled. All cores for solidly grouted reinforced hollow unit masonry shall be filled with grout. For prisms representing partially grouted hollow unit masonry both unfilled and completely filled samples shall be taken and the value of f'_m used for design shall be a weighted average of both as established by the design authority and approved by the Regulatory Agency. The strength of f'_m of each sample shall be taken as the compressive strength of the specimens multiplied by the following correction factor.

Ratio of H/d	2.0	3.0	4.0	5.0
Correction Factor	1.00	1.20	1.30	1.37

where:

H = Height of specimen in inches.

d = Minimum dimension of specimen in inches.

Intermediate values may be interpolated.

(A) STORAGE OF TEST PRISMS. For storage of test prisms follow Method B of ASTM 447 except as modified herein. Test prisms made in the laboratory shall be stored for seven days in air, at a temperature of 70 degrees plus or minus 5 degrees, in a relative humidity exceeding 90 percent; and then in air at a temperature of 70 degrees plus or minus 5 degrees, at a relative humidity of 30 percent to 50 percent until tested.

Test prisms made in the field shall be stored undisturbed and protected from freezing and excessive drying for 48 hours in the field under the same conditions, insofar as possible, and adjacent to the work they are to represent. After field storage, they shall be transported to the laboratory for continued curing as specified for laboratory constructed prisms.

Test prisms and cores cut from the work shall not be taken before the work is 7 days old. Prisms cut from thw work shall be stored as required for prisms made in the field.

(B) SAMPLING, TEST SERIES, AND COMPRESSION TESTS. Not less than five specimens shall be made for each initial preliminary test series required to establish f'_m. Not less than three specimens shall be made for each field test series required to confirm that the materials are as specified in the design.

(C) DETERMINATION OF f'_m. The value of f'_m shall be the average value of all specimens tested but shall not be more than 125 percent of the minimum value determined by tests, whichever is less.

When approved by the Regulatory Agency, tests may be analyzed statistically considering the variability of test results.

12A.8.2 GROUT TEST AND FIELD MORTAR TESTS

Tests for grout and field mortar shall conform to this Section.

(A) GROUT SAMPLES FOR COMPRESSION TESTS. On a flat, nonabsorbent base form a space approximately 3 inches by 3 inches by 6 inches high, i.e., twice as high as it is wide, using masonry units having the same moisture conditions as those being laid. Line the space with a permeable paper or porous separator so that water may pass through the liner into the masonry units. Thoroughly mix or agitate grout to obtain a fully representative mix and place into molds in two layers, and puddle each layer with a 1 inch by 2 inch puddling stick to eliminate air bubbles. Level off and immediately cover molds and keep them damp until taken to the laboratory. After 48 hours set, have the laboratory carefully remove the masonry unit mold and place the grout samples in the fog room until tested in the damp conditions.

(B) FIELD MORTAR SAMPLES FOR CORPRESSION TESTS. Spread a 1/2 inch layer of mortar on masonry units having the same moisture conditions as those being laid. Place a masonry unit on top of the mortar and press to achieve a 3/8 inch mortar joint. After pressing let stand for 2 minutes if the mortar contains 5/8 parts of lime to cement by volume or less; let stand 3 minutes if the mix contains more lime. Immediately remove mortar and place in a 2 inch round by 4 inch high cylinder mold (or a 2 inch cube mold), compressing the mortar using a flat stick or fingers. Lightly tap mold and and level off. Immediately cover mold on opposite sides and keep it damp until taken to the laboratory. After 48 hours, the laboratory shall remove the mortar specimen from the mold and place it in a fog room until tested in the damp condition.

(C) SLUMP TESTS FOR GROUT. Slump tests for grout shall conform to ASTM C143. Substitute the word "grout" for "concrete" in ASTM C143.

(D) COMPRESSION TESTS. Excluding curing, storage, and test age requirements, compression testing procedure for field mortar cubes shall conform to Sec. 8.6.2, 8.6.3, and 9 of ASTM C109. Procedures for field mortar cylinders and for grout shall conform to Sec. A6.3.3 through A.6.3.6, A.6.4, and A.6.5 of ASTM C780.

(E) REQUIRED STRENGTHS. Unless higher strengths are required by the construction documents, minimum required strengths shall be 2000 psi for grout, 1500 psi for field mortar cylinders (and 2000 psi for field mortar cubes).

12A.8.3 CORE TESTS FOR SHEAR BOND

Core tests for shear bond between grout and masonry units used in unreinforced and reinforced grouted masonry construction shall conform to the provisions of this Subsection.

(A) SAMPLES. Samples shall be cores drilled from the wall with axes perpendicular to the face of the wall and diameters approximately 2/3 the wall thickness. These shall contain no reinforcing and shall be taken from locations selected by the design engineer who shall also specify the procedure for repair of the holes in the wall.

(B) NUMBER OF TESTS. A test series shall comprise one test between grout and masonry unit unit for each combination of different grout type and/or masonry unit type. One test series shall be made for each 5,000 square feet of wall or equivalent but not less than one series for any building.

(C) PROCEDURES. The wall shall be at least 14 days old before cores are taken. Cores shall be tested at a minimum of 28 days of age. Storage shall be as required for prisms.

The apparatus shall be of an approved design, similar to a guillotine, designed to shear only one wythe of masonry units from the grout. The shear force and its reaction shall be capable of being applied as close to the bond lines between units and grout as is practicable, one on one side of the plane and the other on the opposite side. Uniform bedding for the shearing force and the reaction shall be provided, both symmetric about a plane which contains the axis of the core. No forces external to the core and perpendicular to the shear plane shall be applied.

Core samples shall not be soaked before testing. The apparatus shall be placed and loaded in a testing machine as required for prisms.

The unit shear strength shall be calculated and reported as the maximum load divided by the shear area. Visual examination of all cores shall be made to ascertain if the joints

12A.8.3 Cont.

are filled. The report shall include the results of these examinations and the conditions of all cores cut on each project regardless of whether or not the core specimens failed during the cutting operation.

Where an unusual number of cores fail during the cutting operation, the design authority shall determine if the test program is extensive enough to satisfy the requirements of Sec. 12A.1.5.

TABLE 12A-1

MAXIMUM RATIO OF HEIGHT TO THICKNESS AND MINIMUM THICKNESS OF MASONRY WALLS

TYPE OF MASONRY	Maximum ratio of unsupported height or length to thickness[1,6]	Maximum nominal thickness (inches)[3,6]
STRUCTURAL WALLS:		
Unburned Clay Masonry	10	16
Stone Masonry	14	16
Cavity Wall Masonry	20[5]	8
Hollow Unit Masonry	20	8
Solid Masonry	20	8[8]
Grouted Masonry	20[2]	6
Reinforced Grouted Masonry	25[2,9]	6
Reinforced Hollow Unit Masonry	25[2,9]	6[7]
NONSTRUCTURAL AND PARTITIONS:[4]		
Unreinforced	36[5]	2
Reinforced	48	4

[1] For cantilever walls, the actual height or length, as applicable, used to compute the actual thickness ratio shall be doubled.

[2] If the only structural function of the wall is the enclosure of a building's exterior, the maximum thickness ratio may be increased to 22 for grouted masonry and 36 for reinforced walls.

[3] The minimum thickness requirements of Sec. 12A.3 shall also be satisfied.

[4] The thickness of plaster coatings may be considered in satisfying thickness ratios and minimum thickness requirements but shall not be used to take stresses.

[5] In determining the thickness ratio for cavity walls, an effective thickness shall be used.

For cavity walls loaded on both wythes the effective thickness for thickness ratio determination only shall be determined from the following formula:

$$T = \sqrt{t_1^2 + t_2^2}$$

where

T = Effective thickness.

t_1, t_2 = Thickness of each wythe.

(For cavity walls loaded on one wythe only, the effective thickness shall be taken for that loaded wythe only.)

See Sec. 12A.6.1(B) for the definition of effective thickness to be used for masonry design.
See Sec. 12A.6.1(E) for applicable cross-sectional areas for masonry design.

[6] The maximum ratio of height or length may be increased and the minimum thickness may be decreased when justified by substantiating data.

[7] Nominal 4 inch loadbearing reinforced hollow clay unit masonry walls with a maximum unsupported height or length to thickness of 27 may be permitted provided net area unit strength exceeds 8000 psi, units are laid in running bond, bar sizes do not exceed 1/2 inch with no more than two bars or one splice in a cell and joints are flush cut, concave, or a protruding v-section. Minimum bar covering where exposed to weather may be 1-1/2 inches.

[8] These thicknesses may be reduced to 6 inches for grouted walls and 8 inches for solid masonry walls in one-story buildings when the wall is not over 9 feet in total height, provided that when gable construction is used an additional 6 feet in height is permitted to the peak of the gable.

[9] Except for masonry reinforced as required, the maximum thickness ratios for one-story walls designed as deep beams may be increased to 36.

TABLE 12A-2

ALLOWABLE WORKING STRESSES IN UNREINFORCED MASONRY

	MORTAR TYPE								
	M	S	M or S				N		
Material[6]	Compression[1]	Compression[1]	Shear or Tension in Flexure[2,3]		Tension in Flexure[3,4,8]		Compression[1]	Shear or Tension in Flexural[2,3,8]	
Special Inspection required	No	No	Yes	No	Yes	No	No	Yes	No
Solid Brick Masonry									
>4501 psi[7]	250	225	20	10	40	20	200	15	7.5
2501-4500 psi[7]	175	160	20	10	40	20	140	15	7.5
1500-2500 psi	125	115	20	10	40	20	100	15	7.5
Solid Concrete Masonry									
Grade N	175	160	12	6	24	12		12	6
Grade S	125	115	12	6	24	12		12	6
Grouted Masonry									
>4501 psi[7]	350	275	25	12.5	50	25			
2501-4500 psi[7]	275	215	25	12.5	50	25			
1500-2500 psi	225	175	25	12.5	50	25			
Hollow Unit Masonry[5]	170	150	12	6	24	12	140	10	5
Cavity Wall Masonry									
Solid Units[5]									
>2501 psi	140	130	12	6	30	15	110	10	5
1500-2500 psi	100	90	12	6	30	15	80	10	5
Hollow Units	70	60	12	6	30	15	50	10	5
Stone Masonry									
Cast Stone	400	360	8	4	--	--	320	8	4
Natural Stone	140	120	8	4	--	--	100	8	4
Unburned Clay Masonry	30	30	8	4	--	--			

[1] Allowable axial or flexural compressive stresses in psi on gross cross-sectional area (except as noted). The allowable working stresses in bearing directly under concentrated loads may be 50 percent greater than these values. Allowable axial stresses are only applicable if the maximum thickness ratios of Table 12A-1 are not exceeded. Reduce these values by 20 percent when designing columns.

[2] This value of tension is based on tension across a bed joint, i.e., vertically in the normal masonry work.

[3] No tension allowed in stacked bond across head joints.

[4] The values shown here are for tension in masonry in the direction of the bond, i.e., horizontally between supports.

[5] Net bedded area or net cross-sectional area, whichever is more critical.

[6] Strengths listed in this column are those of masonry units.

[7] When the required strengths of the units exceed 2500 psi, compression tests of the units conforming to the applicable reference documents and Sec. 12A.7 shall be made. This shall not be required if certification acceptable to the Regulatory Agency accompany the units.

[8] Allowable shear and tension stresses where lightweight concrete units are used are limited to 85 percent of the tabulated values.

TABLE 12A-3

ASSUMED COMPRESSIVE STRENGTH OF MASONRY
f'_m -psi

Type of Unit	Compressive Strength of Units, psi or Grade	f'_m [3]		
		Type N Mortar	Type S Mortar	Type M Mortar
Solid Clay[4] and Net Area of Hollow Clay	14,000 psi gross[1]	4,300[2,5]	5,300[2,5]	6,300[2,5]
	12,000 psi gross[1]	3,800[2,5]	4,600[2,5]	5,500[2,5]
	10,000 psi gross[1]	3,300[2,5]	4,000[2,5]	4,600[2,5]
	8,000 psi gross[1]	2,700[2,5]	3,300[2,5]	3,800[2,5]
	6,000 psi gross[1]	2,200[5]	2,600[5]	3,000[5]
	4,000 psi gross[1]	1,600	1,900	2,200[5]
	2,000 psi gross[1]	1,100	1,200	1,300
Solid Concrete and Net Areas of Hollow Concrete	6,000 psi gross	1,350	2,400	2,400
	4,000 psi gross	1,250	2,000	2,000
	2,500 psi gross	1,100	1,550	1,550
	1,500 psi gross	875	1,150	1,150
	1,000 psi gross	700	900	900

[1] When the required strength of the units exceeds 3000 psi, compression tests of the units conforming to the applicable reference documents and Sec. 12A.7 shall be made. These tests shall not be required if certification conforming to Sec. 12A.7 and Sec. 12A.8 and acceptable to the Regulatory Agency are provided during construction.

[2] When the assumed f'_m exceeds 2600 psi, prism tests conforming to Sec. 12A.7 and Sec. 12A.8 shall be provided during construction. Certification of the units is not acceptable in lieu of tests.

[3] Intermediate values may be interpolated.

[4] When the alternate design procedure for unreinforced brick masonry of Sec. 12A.6.2 is used for design the units shall comply with the dimension and distortion tolerances specified for type FBS. Where such brick do not comply with these requirements, the compressive strength of brick masonry shall be determined by prism tests as required by Sec. 12A.5.1(A)1.

[5] Where grouted construction is used, the value of f'_m shall not exceed the compressive strength of the grout unless prism tests conforming to Sec. 12A.7 and 12A.8 are provided during construction. As an alternative, the grout strength may be specified at not less than the value of f'_m with grout tests conforming to Sec. 12A.7 and 12A.8 provided during construction for verification.

TABLE 12A-4

ALLOWABLE WORKING STRESSES (PSI) FOR REINFORCED MASONRY

	Reinforced Grouted and Hollow Unit Masonry Special Inspection Required	
Type of Stress	Yes	No
Compression-Axial Walls	See Sec. 12A.6.3(E)	One-half of the values permitted under Section 12A.6.3(E)
Compression-Axial Columns	See Sec. 12A.6.3(F)	One-half of the values permitted under Section 12A.6.3(E)
Compression-Flexural	0.33 f'_m but not to exceed 2000	0.166 f'_m but not to exceed 1000
Shear:		
Reinforcement taking no shear[3] Flexural[2]	1.1 $\sqrt{f'_m}$ 50 Max.	25
Shear walls[3,4]		
M/VD $\geq$ 1[6]	.9 $\sqrt{f'_m}$ 40 Max.	20
M/Vd = 0[6]	2.0 $\sqrt{f'_m}$ 50 Max.	25
Reinforcing taking all shear Flexural	3.0 $\sqrt{f'_m}$ 150 Max.	75
Shear walls[4]		
M/Vd $\geq$ 1[6]	1.5 $\sqrt{f'_m}$ 75 Max.	35
M/Vd = 0[6]	2.0 $\sqrt{f'_m}$ 120 Max.	60
Modulus of Elasticity	600 f'_m but not to exceed 3,000,000	500 f'_m but not to exceed 1,500,000
Modulus of Rigidity	240 f'_m but not to exceed 1,200,000	200 f'_m but not to exceed 600,000
Bearing on Full Area[5]	0.25 f'_m but not to exceed 1,500	0.125 f'_m but not to exceed 750
Bearing on 1/3 or Less of Area	0.30 f'_m but not to exceed 1,800	0.15 f'_m but not to exceed 900

[1] Stresses for hollow unit masonry are based on net section.

[2] Web reinforcement shall be provided to carry the entire shear in excess of 20 psi whenever there is required negative reinforcement for a distance of 1/16 the clear span beyond the point of inflection.

[3] Allowable shear resisted by the masonry where lightweight concrete units are used is limited to 85 percent of the tabulated values.

[4] Interpolate by straight line for M/Vd values between 0 and 1.

[5] This increase shall be permitted only where the least distance between the edges of the loaded and unloaded areas is a minimum of 1/4 of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonable concentric area greater than 1/3, but less than the full area, shall be interpolated between the values given.

[6] M is the maximum bending moment occurring simultaneously with the shear load V at the section under consideration.

TABLE 12A-5

ALLOWABLE SHEAR ON BOLTS[1]

	Unburned Clay Units		All Other Masonry	Shear	
Diameter of Bolts (Inches)	Minimum Embedment (Inches)	Shear (Pounds)[2]	Minimum Embedment (Inches)	Solid Masonry (Pounds)[2]	Grouted Construction (Pounds)[2]
1/4	--	--	4	--	180
3/8	--	--	4	--	270
1/2	--	--	4	230	370
5/8	12	130	4	330	500
3/4	15	200	5	500	730
7/8	18	270	6	670	1000
1	21	330	7	830	1230[3]
1-1/8	24	400	8	1000	1500[3]

[1] Edge distance shall be not less than 2 inches nor 5 bolt diameters for edges parallel to the direction of stress. Edge distances shall be not less than 3 inches nor 6 bolt diameters for edges perpendicular to the direction of stress. Center to center spacings shall be not less than 12 bolt diameters.

[2] The tabulated values for the construction where Special Inspection is not provided. Where Special Inspection is provided 150 percent of these values are permitted.

[3] These values are permitted only with units having a minimum compressive strength of 2500 pounds per square inch or more.

PART C - Amendments to Chapters 1, 2, 3, 4, 5, 7, 8, 9, 10, 11, 13, and 14

Listing of Amendments to Chapters 1, 2, 3, 4, 5, 7, 8, 9, 10, 11, 13, 14

Section with Modification[1]		BSSC Ballot Number
Chapter 1	TITLE	9/1
1.4.1	SEISMICITY INDEX AND DESIGN GROUND MOTIONS	1/1
1.6.3	SPECIAL TESTING	4/1
2.1	DEFINITIONS	2/1, 2/18
2.2	SYMBOLS	2/17, 2/2, 2/3
3.1	DESIGN BASIS	2/4
3.2.1	SOIL PROFILE TYPE	2/5, 3/1
3.2.3	SOIL-STRUCTURE INTERACTION	2/6
3.3.1	CLASSIFICATION OF FRAMING SYSTEMS	2/7
3.3.1	COMMENTARY TO 3.3.1	2/25
3.3.2	COMBINATIONS OF FRAMING SYSTEMS	2/8
3.3.4	SEISMIC PERFORMANCE CATEGORY C	2/19, 2/9
3.5	ANALYSIS PROCEDURES	2/20
3.5	COMMENTARY TO 3.5	2/24
3.6.2	SEISMIC PERFORMANCE CATEGORY B	2/11
3.6.3	COMMENTARY TO 3.6.3 SEISMIC PERFORMANCE CATEGORY C	4/10
3.7.2	ORTHOGONAL EFFECTS	2/10
3.7.2	COMMENTARY TO 3.7.2	4/11
3.7.3	DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEM	2/11
3.7.4	NONREDUNDANT SYSTEMS	2/12
3.7.5	TIES AND CONTINUITY	2/13
3.7.9	DIAPHRAGMS	2/14
3.7.12	VERTICAL SEISMIC MOTIONS FOR BUILDINGS ASSIGNED TO CATEGORIES C AND D	4/3
3.8	DEFLECTION AND DRIFT LIMITS	2/21
4.2	SEISMIC BASE SHEAR	2/15
4.5	OVERTURNING	2/16
4.6.2	P-DELTA EFFECTS	2/17, 2/23
4.6.2	COMMENTARY TO 4.6.2	2/26
5.5	MODAL BASE SHEAR	2/27

[1] Chapter 6 had no modifications.

Section with Modification		BSSC Ballot Number
7.2.2	SOIL CAPACITIES	3/2
7.4.4	SPECIAL PILE REQUIREMENTS	4/4, 5/6, 5/7
7.5.2	FOUNDATION TIES	3/3
7.5.3	SPECIAL PILE REQUIREMENTS	3/5, 5/8, 4/7
8.1	GENERAL REQUIREMENTS	8/5
8.2.2	FORCES	4/8
8.2.5	OUT-OF-PLANE BENDING	8/1
8.2.5	COMMENTARY TO 8.2.5	8/2
8.4	ELEVATOR DESIGN REQUIREMENTS	8/6
Table 8-B		8/3, 8/4, 8/7
Table 8-C		8/8
9.1	REFERENCE DOCUMENTS	7/2, 7/1
9.2	STRENGTH OF MEMBERS AND CONNECTIONS	7/4, 7/3
9.4.1	DETAILING REQUIREMENTS	7/5
9.5.3	DETAILING REQUIREMENTS	7/7
9.6.3	DIAPHRAGM LIMITATIONS	7/8
9.7.1	WALL FRAMING AND CONNECTIONS	7/10
9.7.3	ACCEPTABLE TYPES OF WALL SHEATHING	7/11
Table 9-1		7/12
Table 9-2		7/13
10.2	STRENGTH OF MEMBERS AND CONNECTIONS	6/1
10.2.1	STRUCTURAL STEEL	6/2
10.4.1	ORDINARY MOMENT FRAMES	6/3
10.4.2	SPACE FRAMES	6/3
10.5.1	SPECIAL MOMENT FRAMES	6/4
10.6.5	SPECIAL MOMENT FRAME REQUIREMENTS	6/6
11.1	REFERENCE DOCUMENTS	4/13
11.2	STRENGTH OF MEMBERS AND CONNECTIONS	4/14
11.5.1	MATERIAL REQUIREMENTS	4/16
11.5.1	COMMENTARY TO 11.5.1	4/19
11.6.1	FLEXURAL MEMBERS	4/18
11.8.2	DIAPHRAGM DETAILS AND LIMITATIONS	4/17
13.1.1	IDENTIFICATION OF BUILDING REQUIRING EVALUATION	9/2
14.6	WOOD	7/15

CHAPTER 1

GENERAL PROVISIONS

Change of title for Chapter 1 from "Administration" to "General Provisions".

1.4.1 SEISMICITY INDEX AND DESIGN GROUND MOTIONS

The design ground motions referred to in Part 1 should be defined in terms of Effective Peak Accleration or Effective Peak Velocity-Related Acceleration and should be represented by contour maps (in percent gravity) rather than by counties.

This change does not affect the trial designs so new maps have not been prepared. Such maps will not be needed until a final set of provisions is prepared.

1.6.3 SPECIAL TESTING

Alter the sentence under the first EXCEPTION to read as follows:

EXCEPTION:
Certified mill tests may be accepted for ASTM A706 and, where no welding is required, for ASTM A615 reinforcing steel.

CHAPTER 2

DEFINITIONS AND SYMBOLS

Sec. 2.1 DEFINITIONS

Add the following sentence immediately following the definition of SNOW LOAD.

SNOW LOAD is a vertical load due to the weight of the accumulation of snow. For use in combination with seismic forces an effective snow load shall be used which shall be equal to either 70 percent of the full snow load, or, where conditions warrant and when approved by the Regulatory Agency, not less than 20 percent of the full snow load.

> EXCEPTION:
> Where snow load is less than 30 pounds per square foot, no part of the load need be included in seismic loading.

Change the definition of SHEAR PANEL to read as follows:

SHEAR PANEL is a floor, roof, or wall component sheathed to act a a Shear Wall or Diaphragm.

Sec. 2.2 SYMBOLS

Change the definitions to read as follows:

P_x = The total unfactored vertical design load at and above level x.

Q_L = The effect of live load, reduced as permitted in Section 2.1.

Q_S = The effect of snow load, reduced as permitted in Section 2.1

CHAPTER 3

STRUCTURAL DESIGN REQUIREMENTS

Sec. 3.1 DESIGN BASIS

Change the first paragraph to read as follow:

The requirements of this Chapter shall control the selection of the seismic analysis and design procedures to be used in the design of buildings and their components. The design seismic forces, and their distribution over the height of the building, shall be established in accordance with the procedures in Chapter 4 or Chapter 5; the corresponding internal forces in the members of the building shall be determined using a linearly elastic model. An approved alternate procedure may be used to establish the seismic forces and their distribution; the corresponding internal forces and deformations in the members shall be determined using a model consistent with the procedure adopted. Individual members shall be sized for the shears, axial forces, and moments determined in accordance with these provisions, and connections shall develop the strength of the connected members or the forces indicated above. The deformation of the building shall not exceed the prescribed limits when the building is subjected to the design seismic forces.

3.2.1 SOIL PROFILE TYPES

Change the first subparagraph under soil profile type S_1 to read as follows:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2500 feet per second or by other appropriate means of classification, or

Change the last paragraph to read as follows:

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Proile S_2 or Soil Profile S_3 shall be used depending on whichever soil profile type results in the higher value of seismic coefficient, C_s, as determined in Section 4.2.1.

3.2.3 SOIL-STRUCTURE INTERACTION

Change to read as follows:

The base shear, story shears, overturning moments, and deflections determined in Chapter 4 or Chapter 5 may be modified in accordance with procedures set forth in Chapter 6 to account for the effects of soil-structure interaction.

3.3.1 CLASSIFICATION OF FRAMING SYSTEMS

Change to read as follows:

Each building, or portion thereof, shall be classified as one of the four general framing systems types of Table 3-B. The response modification factor, R, and the deflection amplification factor, C_d, are given in Table 3-B and are used in determining the base shear and the design story drift. Inverted pendulum-type structures associated with buildings are included in Table 3-B.

Commentary to 3.3.1

A large table of framing systems is to be developed and inserted in the commentary with an indication of where each system would fall in Table 3-B.

3.3.2 COMBINATIONS OF FRAMING SYSTEMS

Change Paragraph (A) to read as follows:

(A) R VALUE. The value of R in the direction under consideration at any level shall not exceed the lowest value of R obtained from Table 3-B for the seismic resisting system in the same direction considered above that level.

3.3.4 SEISMIC PERFORMANCE CATEGORY C

Change Subparagraph (A)3 to read as follows:

3. A system with structural steel or cast-in-place concrete braced frames or shear walls in which there are braced frames or shear walls so arranged that braced frames or shear walls in one plane resist no more than the following proportion of the seismic design force in each direction, including torsional effects:

a. Sixty (60) percent when the braced frames or shear walls are arranged only on the perimeter.

b. Forty (40) percent when some of the braced frames or shear walls are arranged on the perimeter.

c. Thirty (30) percent for other arrangements.

This system is limited to 240 feet in height.

Change Paragraph (C) to read as follows:

(C) DEFORMATIONAL COMPATIBILITY. Every structural component not included in the seismic force resisting system in the direction under consideration shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the design story drift, as determined in accordance with Sec. 4.6.

Sec. 3.5 ANALYSIS PROCEDURES

Change to read as follows:

This section prescribes the minimum analysis procedure to be followed. An alternate generally accepted procedure, including the use of an approved site specific spectrum, if desired, may be used in lieu of the minimum applicable procedure. The limitations upon the base shear stated in Section 5.8 apply to any such analysis.

Commentary to 3.5

Insert the following fourth paragraph in the Commentary on page 342.

It is possible with presently available computer programs to perform two dimensional inelastic analyses of reasonably symmetric structures. The intent of such analyses could be to estimate the sequence in which components become inelastic and to indicate those components requiring strength adjustments so as to remain within the required ductility limits. It should be emphasized that with the present state-of-the-art in inelastic analysis there is no one method that can be applied to all types of buildings, and further the reliability of the analytical results are sensitive to:

1. the number and appropriateness of the time-histories of input motion

2. the practical limitations of mathematical modeling including interacting effects of nonstructural elements

3. the nonlinear algorithms

4. the assumed hysteretic behavior

Because of these sensitivities and limitations the maximum base shear produced in the inelastic analysis should be not less than that required by Chapter 5 (Modal Analysis).

3.6.2 SEISMIC PERFORMANCE CATEGORY B

Change 3.6.2(A) to read as follows:

(A) COMPONENTS. Components of the seismic resisting system and other structural components shall conform to the requirements of Sec. 3.7 (except Sec. 3.7.3 and 3.7.12) and to Sec. 7.4.

3.6.3 SEISMIC PERFORMANCE CATEGORY C

Commentary to 3.6.3

The loading is cyclical, so static ultimate load capacities may not be reached. If the combination of these loads and deformations results in stresses below yield, it can be assumed that the system is capable of supporting the gravity loads. If the stresses are above yield, then sufficient ductility under cyclic loading must be provided. If the gravity load-bearing system is to provide any calculated resistance to the seismic resisting system (no matter how small), then the detailing for ductility must be consistent with the values given in Table 3-B. In the example of the flat plate warehouse, the connections can still carry the design gravity loadings if they satisfy the requirements of Section 11.4.1.

3.7.2 ORTHOGONAL EFFECTS

Change to read as follows:

In buildings assigned to Category B, the design seismic forces may be applied separately in each of two orthogonal directions. In buildings assigned to Categories C and D, the critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used.

EXCEPTION:
Diaphragms and components of the seismic resisting system utilized in only one of two orthogonal directions need not be designed for the combined effects.

Commentary to 3.7.2

Add the following sentence to the second paragraph:

For two-way slabs orthogonal effects at slab-to-column connections can be neglected provided the moment transferred in the minor direction does not exceed 30 percent of that transferred in the orthogonal direction and there is adequate reinforcement within lines one and one-half times the slab thickness either side of the column to transfer all the minor direction moment.

3.7.3 DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEM

Change to read as follows:

For buildings assigned to Seismic Performance Categories C or D, the design of a building shall consider the potential adverse effects when the ratio of the strength provided in any story to the strength required is significantly less than that ratio for the story immediately above and the strengths shall be adjusted to compensate for this effect.

3.7.4 NONREDUNDANT SYSTEMS

Change to read as follows:

The design of a building shall consider the potentially adverse effect that the failure of a single member, connection, or component of the seismic resisting system would have on the stability of the building.

3.7.5 TIES AND CONTINUITY

Change the first paragraph to read as follows:

All parts of the building between separation joints shall be interconnected and the connections shall be capable of transmitting the seismic force, F_p, induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist $A_v/3$ times the weight of the smaller portion but not less than 5 percent of the portion's weight.

3.7.9 DIAPHRAGMS

Change the third paragraph to read:

A minimum force equal to 0.5 A_v times the weight of the diaphragm and other elements of the building attached thereto plus the portion of the seismic shear force at that level, V_x, required to be transferred to the components of the vertical seismic resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

3.7.12 VERTICAL SEISMIC MOTIONS FOR BUILDINGS ASSIGNED TO CATEGORIES C AND D

Change to read as follows:

The vertical component of earthquake motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. For horizontal cantilever components, these effects may be satisfied by designing for a net upward force of 0.2 Q_D.

Sec. 3.8 DEFLECTION AND DRIFT LIMITS

Change the last paragraph to read as follows:

The design story drift, Δ, as determined in Sec. 4.6 or from Sec. 5.8 shall not exceed the allowable story drift, Δ_a, obtained from Table 3-C for any story. Single story buildings in Seismic Hazard Exposure Group I that are constructed with non-brittle finishes and whose seismic resisting system is not attached to equipment or processes need not meet the drift requirement in Table 3-C.

Sec. 4.2 SEISMIC BASE SHEAR

Change the last paragraph to read as follows:

The value of C_s, may be determined in accordance with Formulas 4-2, 4-3, or 4-3a, as appropriate. Formula 4-2 requires calculation of the fundamental period of the building as specified in Sec. 4.2.2. For low buildings, or in other instances when it is not desired to calculate the period of the building, C_s shall be determined using Formula 4-3 or 4-3a, as appropriate.

Sec. 4.5 OVERTURNING

Change the last paragraph to read as follows:

The foundations of buildings, except inverted pendulum structures, may be designed for the foundation overturning design moment, M_f, at the foundation-soil interface determined using Formula 4-8 with $\kappa = 0.75$ for all building heights.

4.6.2 P-DELTA EFFECTS

Change the definition of P_x to read as follows:

P_x = the total unfactored vertical design load at and above level x.

Revise the last paragraph to read as follows:

When θ is greater than 0.10, the incremental factor related to P-delta effects, a_d, shall be determined by rational analysis (see Commentary). The design story drift determined in Section 4.6.1 shall be multiplied by the factor $\left(\frac{0.9}{1-\theta} \geq 1.0\right)$ to obtain the story drift including P-delta effect. The increase in story shears and moments resulting from the increase in story drift shall be added to the corresponding quantities determined without consideration of the P-delta effect.

Commentary to 4.6.2

The last paragraph on page 368 should be considered as part of the acceptable P-delta analysis referred to on page 367.

5.5 MODAL BASE SHEAR

Commentary to 5.5:

A plot of spectral coefficients for R and A_v will be developed for inclusion in the commentary.

FOUNDATION DESIGN REQUIREMENTS

7.2.2 SOIL CAPACITIES

Change to read as follows:

The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier or caisson and the soil shall be sufficient to support the structure with all prescribed loads, without seismic forces, taking due account of the settlement that the structure can withstand. For the load combination including earthquake as specified in Section 3.7, the soil capacities must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil.

7.4.4 SPECIAL PILE REQUIREMENTS

Change the second paragraph to read as follows:

The piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in Chapter 11. The pile cap connection may be made by the use of field-placed dowels anchored in the concrete pile. For deformed bars the development length is for compression without reduction in length for excess area. Where special reinforcement at the top of the pile is required, alternative meausures for containing concrete and maintaining ductility at the top of the pile will be permitted provided due consideration is given to forcing the hinge to occur in the contained section.

Change 7.4.4(E) to read as follows:

(E) PRECAST-PRESTRESSED PILES. The upper 2 feet of the pile shall have No. 3 ties minimum at not over 4 inch spacing, or equivalent spirals. The pile cap connection may be by means of dowels as required in Section 7.4.4(C).

Pile cap connection may be by means of developing pile reinforcing strand if a ductile connection is provided.

7.5.2 FOUNDATION TIES

Change to read as follows:

Individual spread footings unless founded directly on rock, as defined in Section 3.2.1(1), shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by other approved means.

7.5.3 SPECIAL PILE REQUIREMENTS

Section 7.5.3(c) should be revised to read:

(C) PRECAST CONCRETE PILES. Ties in precast concrete piles shall conform to the requirements of Section 11.6.2 for the top half of the pile. Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from a dynamic analysis of the soil profile. Pile cap connection shall not be made by developing exposed strand.

Insert the following after Section 7.5.3(D):

(E) PRECAST-PRESTRESSED PILES

1. For the body of fully embedded foundation piling subjected to vertical loads only, or where the design bending moment does not exceed 0.20 M_{nb} (where M_{nb} is the unfactored ultimate moment capacity at balanced strain conditions as defined in Reference 11.1, Section 10.3.2), spiral reinforcing shall be provided such that $\rho_s \geq 0.006$ (0.2 percent).

2. For free standing piling and hollow core or marine piling subject to severe installation and operational forces, spiral reinforcing shall be provided such that $\rho_s \geq 0.022$ (0.7 percent), or a spacing satisfying the following relationship, if it results in a percentage of spiral greater than that given above:

$$S_{sp} = \frac{f_y A_{sp}}{(C + 7\ d_b)\ f_r}$$

where:

S_{sp} = spacing of spiral reinforcing

f_y = yield strength of spiral reinforcing

A_{sp} = area of spiral reinforcing

C = cover over the spiral reinforcing

d_b = diameter of spiral reinforcing

f_r = modulus of rupture of concrete

ρ_s = ratio of volume of spiral reinforcing to total volume of core (out-to-out of spirals) and not less than that given in Section 11.7.2(C).

3. Any piling installed in layered soils imposing severe curvatures during earthquake shall have the same amount of spiral reinforcing indicated in item (2) above, accompanied by additional amounts of flexural reinforcing indicated by moment-curvature relationships developed for the pile and soil profile present.

4. The top and bottom portion of hollow core piling and rigid frame piling where high values of shear and moment occur simultaneously should contain spiral reinforcing with $\rho_s \geq 0.031$ (1.0 percent) for a distance of 2 pile diameter, or 2 times the width of the pile.

CHAPTER 8

ARCHITECTURAL, MECHANICAL AND ELECTRICAL COMPONENTS AND SYSTEMS

Sec. 8.1 GENERAL REQUIREMENTS

New exception to be added to Section 8.1:

EXCEPTIONS:

3. Elevator systems which are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with a Seismicity Index of 1 or 2 or which are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with Seismicity Index of 1 are not subject to the provisions of this Chapter.

8.2.2 FORCES

Add the following sentence immediately after the definition of P and just prior to EXCEPTIONS:

The force, F_p, shall be applied independently vertically, longitudinally, and laterally in combination with the static load of the element.

8.2.5 OUT-OF-PLANE BENDING

Change to read as follows:

Transverse or out-of-plane bending or deformation of a component or system which is subjected to forces as determined in Formula 8-1 shall not exceed the deflection capability of the component or system.

Commentary to 8.2.5

Change to read as follows:

Most walls are subject to out-of-plane forces when a building is subjected to an earthquake. These forces and the bending they induce must be considered in the design of wall panels. This is particularly important for systems composed of brittle materials and/or low flexural strength materials. The conventional limits based upon deflection as a proportion of the span may be used with the applied force as derived from Formula 8-1 and Table 8-B.

New Section to be added:

8.4 ELEVATOR DESIGN REQUIREMENTS

8.4.1 REFERENCE DOCUMENT

The design and construction of elevators and components shall conform to the requirements of ANSI A17.1, American National Standard Safety Code for Elevators, Dumbwaiters, Escalators, and Moving Walks, and the proposed A17 Seismic Regulations, except as modified by provisions of this Chapter.

8.4.2 ELEVATORS AND HOISTWAY STRUCTURAL SYSTEM

Elevators and hoistway structural systems shall be designed to resist seismic forces in accordance with Formula 8-1 and Table 8-B.

W_C is defined as follows:

Element	W_C
Traction Car	C + .4L
Counterweight	W
Hydraulic	C + .4L + .25P

C is the weight of the car
L is rated capacity
W is weight of counterweight
P is weight of plunger

8.4.3 ELEVATOR MACHINERY AND CONTROLLER ANCHORAGE(S)

Elevator machinery and controller anchorages shall be designed to resist seismic forces in accordance with Formula 8-2 and Table 8-C.

8.4.4 SEISMIC CONTROLS

All elevators with a speed of 150 fpm or greater shall be furnished with signaling devices as follows:

(a) A seismic switch device to provide an electrical alert or command for the safe automatic emergency operation of the elevator system.

(b) A counterweight displacement or derailment device to detect lateral motion of the counterweight.

A continuous signal from (b) or a combination of signals from (a) and (b) will initiate automatic emergency shutdown of the elevator system.

8.4.5 RETAINER PLATES

Retainer plates are required at the top and bottom of the car and counterweight except where safety stopping devices are provided. The depth of engagement with the rail shall not be less than the side running face of the rail.

8.4.6 DEFLECTION CRITERIA

The maximum deflection of guide rails, including supports, shall be limited to prevent total disengagement of the guiding members of retainer plates from the guide rails' contact surface.

Table 8-B

Immediately following "Wall Attachments" and indented therefrom, insert Connector Fasteners *with a corresponding* C_c *Factor of* 6.0.

Under the category "Appendages," change "Veneers" to "Veneer Attachments."

Change entry under Partitions = "Elevators and Shafts" to Elevator Shafts.

Add the following new entry:

Architectural Components	C_c Factor	III	II	I
Elevator and Hoistway Structural Systems				
- Structural frame providing the supports for guiderail brackets	1.25	S	G	G
- Guiderails and brackets	1.25	S	G	G
- Car and counterweight guiding members	1.25	S	G	G

Change footnote 4 to read:

Shall be raised one performance level if the area facing the exterior wall is normally accessible within a distance of 10 feet plus one foot for each floor height.

Table 8-C

Add the following new entry:

Mechanical/Electrical Components	C_c Factor	III	II	I
Elevator Machinery and Controller Anchorage	1.25	S	G	G

CHAPTER 9

WOOD

Sec. 9.1 REFERENCE DOCUMENTS

Change Reference 9.12 to read:

Ref. 9.12 One- and Two-family Dwelling Code, 1975
International Conference on Building Officials
Building Officials and Code Administrators
Southern Building Code Congress

Add new references:

9.15 Plywood Design Specifications, APA, 1978
9.16 Plywood Diaphragm Construction, APA, 1978

Sec. 9.2 STRENGTH OF MEMBERS AND CONNECTIONS

Revise the tabulation of strength reduction factors as follows:

The value of the capacity reduction factor, ϕ, shall be as follows:

All stresses in wood members	$\phi = 1.0$
Bolts and other timber connectors not listed below	$\phi = 1.0$
Shear on carriage bolts not having washers under the head	$\phi = 0.67$
Lag screws and wood screws	$\phi = 0.90$
Shear on diaphragms and shear walls as given in this chapter	$\phi = 0.85$

9.4.1 DETAILING REQUIREMENTS

Delete Subsection 9.4.1(c) "ECCENTRIC JOINTS"

9.5.3 DETAILING REQUIREMENTS

Remove Subsection 9.5.3(B) "PLYWOOD SHEAR PANELS' and transfer it to Section 9.6.3.

9.6.3 DIAPHRAGM LIMITATIONS

The existing sentence in Subsection 9.5.3(B), without the heading, should become the first sentence in Section 9.6.3.

Change to read as follows (see 9.5.3):

Plywood used for shear panels which are a part of the seismic resisting system shall be applied directly to the framing members, except that plywood may be used as a diaphragm when nailed over solid lumber planking or laminated decks. The allowable working stress shear for vertical plywood shear walls, used to resist horizontal forces in buildings with masonry or reinforced concrete walls, shall be one-half of the allowable values set forth in Table 9-2.

9.7.1 WALL FRAMING AND CONNECTIONS

Change Paragraph C to read as follows:

(C) BOTTOM PLATES. Studs shall have full bearing on a plate or sill of not less than 2 inch nominal thickness and having a width at least equal to the width of the studs unless specifically excepted in Section 9.7.3.

9.7.3 ACCEPTABLE TYPES OF WALL SHEATHING

Change Paragraph B to read as follows:

(B) PLYWOOD. Plywood panels with a thickness of not less than 5/16 inch for 16 inch stud spacing and not less than 3/8 inch for 24 inch stud spacing. Blocking need not be provided at horizontal joints.

Table 9-1

Change the table heading to read:

ALLOWABLE SHEAR IN POUNDS PER SQUARE FOOT FOR HORIZONTAL PLYWOOD DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

The entry under 10d nails should be corrected from:

3/8" *to* 5/8"

Revise Footnote 1 as follows:

[1] Space nails 10 inches on center for floors and 12 inches on center for roofs along the intermediate framing members. Allowable shear values for nails in framing member of other species set forth in Table 8.1A NDS (Ref. 1) shall be calculated for all grades by multiplying the values for nails in STRUCTURAL I by the following factors: Group III, 0.82 and Group IV, 0.65.

Change the wording under the column heading "BLOCKED DIAPHRAGMS" to read:

Nail spacing at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6).

Table 9-2

Change the table heading to read:

ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SEHAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

Change Column 1 under "Plywood Grade" to read:

C-D, C-C
STRUCTURAL II and
other grades covered
in U.B.C. Standard
No. 25-9

Plywood panel
siding in grades
covered in U.B.C.
Standard No. 25-9

Change Footnotes 1, 2, and 3 to read as follows:

[1] All panel edges backed with 2 inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8 inch plywood installed with face grain parallel to studs spaced 24 inches on center and 12 inches on center for other conditions and plywood thicknesses.

Allowable shear values for nails in framing members of other species set forth in Table 8.1A NDS (Ref. 1) shall be calculated for all grades by multiplying the values for common and galvanized box nails in STRUCTURAL I and galvanized casing nails in other grades by the following factors: Group III, 0.82 and Group IV, 0.65.

2 Reduce tabulated allowable shears 10 percent when boundary members provide less than 3 inch nominal nailing surface.

3 The values for 3/8 inch thick plywood applied direct to framing may be increased 20 percent, provided studs are spaced a maximum of 16 inches on center or plywood is applied with face grain across studs or if the plywood thickness is increased to 1/2 inch or greater.

CHAPTER 10

STEEL

Sec. 10.2 STRENGTH OF MEMBERS AND CONNECTIONS

Change the seventh and eight line to read as follows:

The value of ϕ shall be as follows:

Members and connections which develop the strength of the members or structural systems.	$\phi = 0.90$
Connections which do not develop the strength of the member or structural systems or do not conform to Sec. 10.6.1(A)6	$\phi = 0.67$
Partial penetration welds in columns when subjected to tension stresses	$\phi = 0.80$

10.2.1 STRUCTURAL STEEL

Delete 10.2.1(B)

Change present 10.2.1(C) and 10.2.1(D) to 10.2.1(B) and 10.2.1(C), respectively.

Add new 10.2.1(D).

(E) In AISC specifications 2.5, substitute $v_u \leq 0.68$ in lieu of $V_u \leq 0.55$.

10.4.1 ORDINARY MOMENT FRAMES

Delete Sec. 10.4.1

10.4.2 SPACE FRAMES

Change Sec. 10.4.2 to Sec. 10.4.1 and to read as follows:

Ordinary moment frames, space frames in building frame systems, and space frames incorporated in bearing wall systems shall be designed and constructed in accordance with Ref. 10.1, Part 1 or Ref. 10.2 or Ref. 10.3.

10.5.1 SPECIAL MOMENT FRAMES

Change the "EXCEPTION" to read as follows:

1. Moment frames in one- and two-story buildings assigned to Seismic Performance Category C may be Ordinary Moment Frames.
2. Moment frames in one-story buildings assigned to Seismic Performance Category D may be Ordinary Moment Frames.

Sec. 10.6.5 SPECIAL MOMENT FRAME REQUIREMENTS

Change the first line after the equation in modification 5 to read as follows:

in place of Equation 1.15-2 of Ref. 10.1.

CHAPTER 11

REINFORCED CONCRETE

Sec. 11.1 REFERENCE DOCUMENTS

Change Reference 11.1 to read as follows:

Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-77) excluding Appendix A and replacing Section 9.2.3 with Section 3.7.1 of this document.

Sec. 11.2 STRENGTH OF MEMBERS AND CONNECTIONS

Change the first paragraph to read as follows:

These provisions are based on the use of monolithic cast-in-place reinforced concrete construction. Precast and/or prestressed reinforced concrete components may be used if the resulting construction complies with the requirements of Sec. 3.6 and this Chapter.

11.5.1 MATERIAL REQUIREMENTS

Change the third paragraph to read as follows:

Reinforcement resisting earthquake-induced flexural and axial forces in frame elements and in wall boundary members shall comply with ASTM A706. ASTM A615 Grades 40 and 60 reinforcement may be used in these elements if (a) the actual yield stress based on mill tests does not exceed the specified yield stress by more than 18,000 psi (retests shall not exceed this value by more than an additional 4,000 psi) and (b) the ratio of the actual ultimate tensile stress to the actual tensile yield stress is not less than 1.25.

After Commentary Section 11.5.1, fifth paragraph by including the following sentence at the end of the paragraph:

In the event of a strong earthquake, it is assumed that the structure will undergo reversals of large lateral displacements. It is essential that all structural components be able to accommodate these displacements without critical loss of strength. Even if a particular frame has been designed to support only gravity loads and is not intended to be part of the structural system resisting seismic forces, it must sustain the gravity loads after having been subjected to approximately the same displacements as the seismic resisting system. Therefore, all frame components (which are not designed to resist seismic forces) in Categories C and D buildings are required to have, as a minimum, the details specified in Sec. 11.6. Furthermore, if calculations show that frame components (which are not part of the structural system resisting seismic forces) will have to yield in order to accommodate the calculated displacements of the seismic resisting system, those components must have special transverse reinforcement as specified for Special Moment Frames. The flat plates of flat plate frames of normal proportions and detailed as specified in Section 11.6 will not undergo any significant yield until story drifts greater than those allowable (Table 3-C).

11.6.1 FLEXURAL MEMBERS

Change the second paragraph to read as follows:

At any section of a member subjected to bending the tensile reinforcement ratio, ρ, for the top and for the bottom reinforcement shall not be less than 200/fy nor exceed 0.025 at any section. At least two No. 5 or larger bars shall be provided continuously both top and bottom except in slabs.

Change the sixth paragraph to read as follows:

Web reinforcement perpendicular to the longitudinal reinforcement shall be provided throughout the length of all members except slabs. Stirrups shall have a minimum of two legs and the size shall be not less than No. 3. Maximum spacing of stirrups shall be d/2.

Change the seventh paragraph to read as follows:

Within a distance equal to twice the effective depth from the end of all members except slabs, the amount of web reinforcement shall not be less than given by the following formula:

$$\frac{A_{vd}}{s} = 0.15\ A_s' \text{ or } 0.15\ A_s \tag{11-1}$$

whichever is larger and the spacing shall not exceed d/4. Hoops shall be used as web reinforcement within a distance equal to the effective depth from the end of the member.

Add an eighth paragraph as follows:

Slabs without beams and supported on columns may be used for ordinary moment frames provided those slabs satisfy the requirements of Chapter 13 of Reference 11.1 and this Section. Bottom bar reinforcement, A_s', shall be provided continuous through or anchored within a column and not less than that given by the following formula:

$$A_s' = 2\ \frac{(V-Vp)}{0.85fy} \tag{11-2}$$

where V is the shear force transferred to column due to unfactored gravity loads and Vp is the sum of the vertical components of the forces in any prestressing tendons passing through or anchored within the column. At least two No. 4 or larger bars shall be provided continuous through or anchored within the column in both directions and both top and bottom. In slabs without beams, column strip negative moment reinforcement shall be distributed so that at least 60 percent of the required reinforcement is concentrated within lines one and one-half times the slab thickness either side of the column. The shear stress, v, on a critical section located half the effective depth of the slab from the column perimeter, and caused by the shear force V shall not exceed $2\sqrt{f_c'}$. If there is no spandrel beam at the discontinuous edge of a slab, reinforcement within four slab thicknesses either side of a column face and adjacent to the edge shall be detailed so that it can act effectively as torsion reinforcement considering the possibility of full reversals of the sense of the torsional moments. If the torsional strength of the spandrel beam framing into a column exceeds the flexural strength of the slab at its connection with the beam for the adjacent half panel width, all shear shall be assumed transferred to the column via the beam.

11.8.2 DIAPHRAGM DETAILS AND LIMITATIONS

Change the third paragraph to read as follows:

A cast-in-place topping on a precast floor system may serve as the diaphragm provided the cast-in-place topping is proportioned and detailed to resist the design shear forces under the effects of any loading combination (which could induce tensile or compressive stresses simultaneously to the shear forces). For buildings in performance Categories C and D, alternate techniques based on the use of untopped precast and/or prestressed components of concrete floor systems may be used only if it can be shown by experiments and analysis based on established engineering principles that they will offer the same shear strength, stiffness, stability, durability, and sufficient energy dissipation capacity, as a monolithic cast-in-place ordinarily reinforced concrete diaphragm.

CHAPTER 13

SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

13.1.1 IDENTIFICATION OF BUILDING REQUIRING EVALUATION

Change the word "designed" to "with a permit issuance date" *in paragraphs one and two.*

CHAPTER 14

GUIDELINES FOR REPAIR AND STRENGTHENING OF EXISTING BUILDINGS

Section 14.6 WOOD

Add to the reference documents in the first paragraph:

1) Plywood Design Specification, 1978, APA, and 2) Plywood Diaphragm Construction, 1978, APA.

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This report is also available from the Building Seismic Safety Council as BSSC 82-2.

☐ Document describes a computer program; SF-185, FIPS Software Summary, is attached.

11. ABSTRACT (A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here)

This report presents amendments to the seismic design recommendations contained in the report "Tentative Provisions for the Development of Seismic Regulations for Buildings" developed by the Applied Technology Council. These amendments were prepared in a review project conducted by the Building Seismic Safety Council and the National Bureau of Standards. The amendments plus the Tentative Provisions will be used in a trial design program to provide information for estimating the impact of adopting the recommendations.

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