NBSIR 80-2111-11

Review and Refinement of ATC 3-06 Tentative Seismic Provisions

Joint Committee on Review and Refinement
Coordinating Committee
Technical Committee
Committee 1: Seismic Risk Maps Committee 2: Structural Design Committee 3: Foundations Committee 4: Concrete Committee 5: Masonry Committee 6: Steel Committee 7: Wood Committee 8: Architectural, Mechanical and Electrical Committee 9: Regulatory Use

Report of Joint Committee on Review and Refinement

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 the Tentative Provisions for trial designs

U.S. Department of Commerce
National Bureau of Standards
National Engineering Laboratory
Center for Building Technology
Washington, D.C. 20234

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Errata for NBSIR 80-2111-11

p. A140 - Change 9-0-1 to 87-5
(9-0-1)

p. A141 - Change 8-7-1 to 87-11
(8-7-1)

p. A143 - Change 87/10 to 87-9
(6-3-1) (9-0-1)

- Insert dimension line and arrow heads between paragraphs (C)3.A and (C)3.B

- Insert 87/10 adjacent to paragraph (C)3.B
(6-3-1)

p. A181 - Item 12-4-4: The second sentence "Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel." Should not have been deleted.

p. A184 - Item 5/10: Insert M 15 adjacent to the 12th line

p. A189 - Item 5/12, line (O): Change (K) to (L)

- Item 5/22, line 12.5.2: Insert 12-7N to identify committee ballot item

p. A190 - Insert "12-7N continued" to the left of the first line

p. A192 - Move committee ballot item M-29 to follow the last paragraph on the page.

NBSIR 80-2111-11

REVIEW AND REFINEMENT OF ATC 3-06 TENTATIVE SEISMIC PROVISIONS

REPORT OF JOINT COMMITTEE ON REVIEW AND REFINEMENT

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Prepared for use by the:

BUILDING SEISMIC SAFETY COUNCIL

Sponsored by:

FEDERAL EMERGENCY MANAGEMENT AGENCY

Center for Building Technology
National Bureau of Standards
Washington, D.C. 20234

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U.S. DEPARTMENT OF COMMERCE, Phillip M. Klutznick, Secretary
Luther H. Hodges, Jr., Deputy Secretary
Jordan J. Baruch, Assistant Secretary for Productivity, Technology, and Innovation
NATIONAL BUREAU OF STANDARDS, Ernest Ambler, Director

ABSTRACT

The Tentative Provisions for the Development of Seismic Regulations for Buildings were developed by the Applied Technology Council to present, in one comprehensive document, current state of knowledge pertaining to seismic engineering of buildings. The Tentative Provisions are in the process of being assessed by the building community. This report is one of a series of reports that documents the deliberations of a group of professionals jointly selected by the Building Seismic Safety Council and the National Bureau of Standards and charged with reviewing the Tentative Provisions prior to the conduct of trial designs. The group is divided into nine technical committees, each of which focused on a particular portion of the Tentative Provisions. The nine committees proposed recommendations for change to the parent group, the Joint Committee, through a Coordinating Committee. The Coordinating Committee made some modifications to the technical committees' recommendations to ensure consistency among the recommendations. This report documents the actions of the Joint Committee on the 198 recommendations for change that were presented to it. The first part of the report is a summary of the results, and the appendices contain the full documentation for each recommended change. The actions of each of the nine technical committees is documented in a separate report.

Keywords: Building; building codes; building design; earthquakes; engineering; standards; structural engineering.

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1.0 INTRODUCTION

The Tentative Provisions for the Development of Seismic Regulations were developed by the Applied Technology Council (ATC) in an effort that included a wide range of experts in the actual drafting of the provisions. Two external review drafts were circulated to a large portion of the interested and informed community of eventual users. However, because the Tentative Provisions were innovative, doubts about them existed. Consequently, an attempt was made to investigate these doubts and to improve the Tentative Provisions where possible before an expensive assessment of the Tentative Provisions was undertaken by conducting trial designs.

This review and refinement project was planned and conducted by the National Bureau of Standards (NBS) with the advice and approval of the Building Seismic Safety Council (BSSC), a private sector organization formed in 1979 for the purpose of enhancing public safety by providing a national forum to foster improved seismic safety provisions for use by the building community. The project was conducted in accordance with the Work Plan in Appendix D which was agreed to by NBS and the Board of BSSC.

The assessment of the Tentative Provisions was performed using the committee structure shown on the cover of this report. Nine technical committees were formed with interests that collectively cover the Tentative Provisions. The Joint Committee on Review and Refinement consists of all voting members of the Technical Committees. The chairmen of the Technical Committees form a Coordinating Committee. Membership of each Technical Committee is made up of representatives of organizations that have particular interest in the Tentative Provisions; the participants are listed in Appendix E of this report.

In addition to the voting members, each Technical Committee includes a non-voting member from each of the following organizations: The Applied Technology Council, the Building Seismic Safety Council and the National Bureau of Standards. The ATC representative served as a technical resource to the committee since he was closely involved with the development of the provisions of interest to the committee. The NBS representative was the technical secretary throughout the effort. The BSSC representative provided a link with the Building Seismic Safety Council, which will be involved in trial designs and evaluations.

2.0 SUMMARY OF ACTIVITIES

2.1 Development of the Recommendations for Ballot

The initial meeting for all the committees was held on December 11, 1979, at the National Bureau of Standards. The morning was devoted to a session of the Joint Committee at which the proposed work plan for the project was presented and discussed. The afternoon was devoted to organizational meetings of the nine technical committees.

Following the initial meeting, the technical committees conducted their business by correspondence and meeting (except Committees 1 and 7, which relied entirely on correspondence). The general sequence of activity was as follows:

1. Written proposals for change collected;

2. Public work sessions on the proposals for change and development of a ballot for the committee recommendations;

3. Letter ballots;

4. Additional work sessions (for some committees) to resolve disputes and prepare additional ballots;

5. Submission of recommendations to the Coordinating Committee.

The activities of each committee are fully documented in the report in this series of reports for which the last digit of the report number matches the committee number (e.g. report NBSIR 80-2111-3 documents the activities of Technical Committee 3).

The Coordinating Committee examined problems of overlap and conflict among the recommendations of the nine committees, modifying a few of the recommentions in the process. The final recommendations for ballot by the Joint Committee were then distributed at the July 16-17, 1980 meeting.

2.2 Final Meeting of the Joint Committee

The final meeting of the Joint Committee was held on July 16 and 17 at the National Bureau of Standards. Each committee's recommended changes were discussed in the following format:

1. The committee members (usually the chairman) presented their reasons for the proposed change.

2. For those items with significant dissent within the committee, a committee member presented the minority viewpoint.

3. The ATC representative on the committee presented his opinion of the recommended changes.

4. The audience participated in an open discussion of the committee's recommended changes.

Following is a very brief summary of the discussion and actions taken at the meeting.

Committee 1's two recommendations were presented by the secretary, Bruce Ellingwood. Neville Donovan, the ATC representative, registered opposition to both. Discussion was then opened on the Coordinating Committee's special ballot items concerning reduction of the Seismicitiy Index for Map Areas 1 through 5. E. V. Leyendecker pointed out that the Seismicity Index items were placed on the ballot because of the considerable interest in them by a number of committees. This was not intended to imply endorsement, instead it was intended as a means to place the issue before the full Joint Committee.

Committee 2 used several members to present their recommendations and noted that about six of their 27 recommendations were simple editorial clarifications. Roland Sharpe, the ATC representative, agreed with each of the recommended changes and further noted a desire to see the trial designs incorporate a a test of both the original and the newly recommended drift limits. There was very little discussion.

Committee 3's recommendations were presented by the chairman, Richard Simon. Discussion turned to the chapter on the soil-structure interaction provisions. Committee 3 had originally recommended that the chapter be deleted, but had withdrawn the recommendation within the Coordinating Committee in order to resolve a conflict with a recommendation from Committee 2 (item 2/6 - note: the number preceding the slash identifies the Committee making the recommendation, the number following the slash is the recommendation number for that committee; see section 3 for more information). Henry Degenkolb, the ATC representative, explained that he had not opposed Committee 3's recommendation. Ron Mayes, an ATC representative on another committee, argued that deletion of the entire chapter exceeded the scope of review and refinement committees. The only other item of discussion was that Howard Simpson and a few other members of Committee 2 opposed ballot item 3/1, stating that the proper soil profile category for sites not fitting the description for any of the three types was type S2.

Committee 4's recommendations concerning concrete piles (ballot items 4/4, 4/6, 4/7, S/6, S/7, and S/8) were presented by Neil Hawkins, who acted for the chairman. Loring Wylie, the committee's representative from the Structural Engineers Association of California (SEAOC), opposed item 4/6 in particular, citing the importance of category D buildings and the lack of substantiating data on pile performance in large earthquakes. Richard Simon, the chairman of Committee 3, opposed ballot item 4/6 because it would allow the use of precast-prestressed piles for category D buildings. Hawkins argued for the recommendation, stating that the lateral reinforcement used in such piles in California made them weaker than those in other regions of the country. There was also some discussion of the recommendations concerning the use of exposed strand for connecting precast piles to pile caps.

Frank M. Fuller of Raymond International Builders presented revisions to the provisions for cased and uncased concrete piles in category B and C buildings. One of his objectives was to reverse the present provision which requires more reinforcement in a cased pile than in an uncased pile. His proposed revisions were not identical to any of those proposed by the committees and therefore were not on the ballot for the Joint Committee. His proposals are included in Appendix C of this report.

Hawkins then presented Committee 4's recommendation concerning the use of the March 1980 draft proposed for Appendix A of American Concrete Standard ACI 318 (item 4/12), which elicited considerable discussion. Proponents argued that the draft Appendix A contains several technical improvements and that the economic impact would be large. Opponents were principally concerned that several different issues were covered in the one recommendation, making it an "all-or-nothing" proposition, and that the version of Appendix A being referred to was not a real standard but only a subcommittee draft, which would be subject to future change.

Hawkins next presented Committee 4's other recommendations. Ed Zacher of Committe 2 pointed out that Committee 2 had rejected a proposal similar to item 4/3. Tony Wintz of Committee 8 stated that items 4/8 and 4/9 were within the scope of Committee 8, not Committee 4, and opposed those recommendations because Committee 8 had not had the opportunity to consider them.

Committee 5's recommendations were presented by George Hanson, co-chairman of the committee. Ed Johnson, the SEAOC representative on the committee expressed opposition to the committee's recommendations. Johnson claimed the committee membership was unbalanced in the favor of the masonry industry. Ron Mayes, the ATC representative, stated that the committee needed more time to complete its work. Mayes opposed several of the specific recommendations. The open discussion covered many issues, but one of the most substantial was Howard Simpson's opinion that the committee's grouping of recommendations made it impossible to conscientiously cast a ballot. Simpson gave an example of one ballot item that he felt included five substantive changes, four of which he favored and one of which he opposed. E. V. Leyendecker, the committee secretary indicated that he had attempted to group the numerous ballot items to reduce the total number of ballot items below those considered by the committee.

Committee 6's recommendations were presented by William Sontag, who acted for the chairman. Charles DeMaria, the SEAOC representative, opposed ballot item 6/5, which deals with the maximum allowable axial force in columns of special moment frames. William Smith, an alternate on the committee, presented a set of proposals prepared by the American Iron and Steel Institute for modifying the provisions to include an R factor and special design rules for steel eccentric braced frames. The proposals were not a part of Committee 6's recommendations and therefore were not included on the ballot. They are included in Appendix C of this report.

Committee 7's recommendations were presented by the chairman, Dan Brown. Ed Zacher, the ATC representative expressed opposition to items 7/6, 7/9, and 7/14. A short discussion of item 7/9 followed.

Committee 9's two recommendations were presented by the chairman, Norton Remmer. The only discussion was on the issue of existing buildings, which was unrelated to the recommendations. Ed Pfrang, the chairman of the Joint Committee, stated in response to a question that there were no plans to include existing buildings in the trial designs.

Committee 8's recommendations were presented by the chairman, Robert Sockwell. Tom Wosser, the ATC representative, supported the recommendations and stated that the first four were mostly editorial and that the recommendations for elevators were a real improvement. A question was raised concerning the reference to American National Standard A17.1 in item 8/6. Sockwell responded it was not the intent that A17.1 be ". . .the singular acceptable design standard for elevators. . . [but] the committee considered A17.1 to be the most up-to-date and appropriate standard for reference to <u>seismic design</u> of elevator systems."

Committee 5 then reported back to the Joint Committee with a proposal for a new ballot. The new ballot had been prepared by subdividing the original items into smaller units (more ballot items). They also prepared an accompanying document that included the record of their committee ballot, a description of the effect of the change, and a comment from the ATC representative for each item. They pointed out that the new proposal was merely a repackaging of the previous recommendations in order to facilitate voting, i.e. that they had not changed the substance of their recommendations. Where grouping of items was done, it was with ATC concurrence. Ron Mayes, the ATC representative, told the Joint Committee that he felt the new proposal was a significant improvement and that he agreed with most of the recommendations. The Joint Committee accepted their proposal, which meant that the portion of the original ballot dealing with Committee 5 was to be disregarded and eventually replaced by a supplementary ballot.

Committee 4 then proposed that they be allowed to prepare a supplementary ballot to be distributed along with Committee 5's supplementary ballot. Their proposal was somewhat different, however. The supplementary items would be a detailed breakdown of the technical improvements involved in the recommendation to use the March 1980 draft version for Appendix A of ACI 318 (ballot item 4/12). Furthermore, the supplementary items would not replace any of the original ballot items, but would only be effective if item 4/12 failed to pass the Joint Committee. After some discussion on the propriety of the proposal, the Joint Committee agreed with Committee 4. Committee 4 then identified and prepared seven items for the supplementary ballot.

Roland Sharpe of ATC pointed out that some recommendations for the Joint Committee from a given committee were closely related to proposals in other committees and requested that the results of these deliberations be made known. James Harris of NBS produced a list of such items closely related to issues discussed by Committee 2. It was agreed that pertinent excerpts from the minutes of the various committees would be collected and distributed to the membership of the Joint Committee.

The instructions for the Joint Committee ballot (and the supplementary ballot) including the pertinent excerpts of committee minutes are included in Appendix B of this report.

3.0 JOINT COMMITTEE BALLOT

Tables 1, 2, and 3 indicate the results of the ballots of the Joint Committee. Each item is listed in tables 1, 2 and 3 with an item number and a short title. The first digit in the item number corresponds to the technical committee recommending the change, and the number following the slash ("/") is a consecutive number for the recommendations within that committee. The Coordinating Committee is the final author of those recommendations for which an "S" is the original character of the item number. The complete text, the vote of the sponsoring technical committee, and the reason for the change for each recommendation are included in Appendix A of this report. (In the case of Committee 5, only the final recommendations agreed to at the July 16-17 meeting are included; they replace the original proposals of Committee 5.) The item number is the easiest way to find the particular item in Appendix A, since the items are arranged consecutively in Appendix A. Each recommendation also appears in the individual committee reports referred to in the previous section.

Table 1 contains the 143 items that passed with a 2/3 or greater majority, which is the criterion for passage established at the initiation of the project. Table 2 contains 19 items that did not pass by that criterion but which did have a simple majority of affirmative votes. Table 3 contains 36 items that did not receive a simple majority of affirmative votes. Because of the supplementary ballot for Committees 4 and 5 that was agreed to at the July 16-17 meeting, there were actually two ballots cast. Of the 65 eligible votes, 52 cast the first ballot and 42 cast the second ballot, which dealt only with the recommendations of Committee 5 and the supplementary recommendations of Committee 4.

4.0 DESCRIPTION OF APPENDICES

There are five appendices to this report, which contain the official documents of the Joint Committee. Appendix A contains all of the recommendations that the Joint Committee balloted, whether they were approved or not. The recommendations in Appendix A actually originated in the official documents of the nine technical committees and the Coordinating Committee. The other appendices simply complete the documentation of the Joint Committee's actions. There are no official minutes for the Joint Committee's meetings, although the proceedings of the final meeting are described in section 2.2 of this report.

TABLE 1 ITEMS WITH GREATER THAN 2/3 MAJORITY

ITEM	SHORT TITLE	YES	NO	ABST.
1/1	CONTOUR MAPS	33	16	3
2/1	SNOW LOAD DEFINITION	34	13	5
2/2	GL DEFINITION	47	1	4
2/3	QS DEFINITION	46	2	4
2/4	ALLOW ALTERNATE ANALYTICAL PROCEDURE	48	0	4
2/5	ROCK CLASSIFICATION	47	1	4
2/6	SOIL-STRUCTURE INTERACTION ANALYSIS	45	2	5
2/7	NAME OF INVERTED PENDULUM	46	1	5
2/8	R VALUE FOR MIXED STRUCTURES	47	1	4
2/9	DRIFT COMPATIBILITY	43	3	6
2/10	ORTHOGONAL COMBINATION OF LOADS	42	4	6
2/11	DISCONTINUITIES IN STRENGTH BY STORY	45	1	6
2/12	REDUNDANCY	43	5	4
2/13	TIES AT JOINTS	46	1	5
2/14	NAME OF VX	46	1	5
2/15	USE OF EQUATION FOR CS	46	2	4
2/16	RESULTANT OF OVERTURNING	43	4	5
2/17	PX DEFINITION	46	1	5
2/18	SHEAR PANEL DEFINITION	45	3	4
2/19	HEIGHT LIMITS	45	2	5
2/20	ALLOW ALTERNATE LOAD ANALYSIS	47	1	4
2/21	EXCEPTION TO DRIFT LIMSIT	40	7	5
2/23	CALCULATION OF P-DELTA EFFECT	43	3	6
2/24	COMMENTARY ON ALTERNATE ANALYTICAL PROCEDURE	44	2	6
2/25	COMMENTARY ON CLASSIFICATION OF FRAMING SYSTEMS	36	6	10
2/26	COMMENTARY ON DRIFT AND P-DELTA	42	2	8
2/27	COMMENTARY ON SPECTRAL COEFFICIENTS	39	4	9
3/1	COEFFICIENT S FOR UNKNOWN SOIL PROFILE	38	8	6
3/2	ACCEPTABLE STRAINS IN LIEU OF ELASTIC LIMIT	48	0	4
3/3	TIES BETWEEN SPREAD FOOTINGS	43	5	4
3/5	FLEXURE ON PRECAST PILES	46	2	4
4/1	MILL TESTS FOR REINFORCING STEEL	45	1	6
4/3	VERTICAL ACCELERATION ON PRESTRESSED CONCRETE	32	14	6
4/4	PILE CAP CONNECTION	40	5	7
4/7	CATEGORY C PRECAST-PRESTRESSED PILES	32	13	7
4/8	VERTICAL FORCE ON NONSTRUCTURAL COMPONENTS	33	12	7
4/9	WALL CONNECTION DESIGN COEFFICIENT	34	11	7
4/10	COMMENTARY ON CATEGORY C DETAILS	37	7	8
4/11	COMMENTARY ON ORTHOGONAL EFFECT IN SLABS	38	6	8
4/13	REFERENCE ACI 318-77	33	5	4
4/14	ALLOW PRECAST AND/OR PRESTRESSED CONCRETE	33	6	3
4/16	REINFORCING STEEL	30	9	3
4/17	UNTOPPED PRECAST AND/OR PRESTRESSED COMPONENTS	31	8	3
4/18	FLAT SLAB CONSTRUCTION	35	4	3
4/19	COMMENTARY ON FLAT PLATE FRAMES	31	8	3
5/1	BACKGROUND	39	1	2
5/2	STRENGTH	39	1	2
5/3	DESIGN PROCEDURES	38	3	1
5/4	S. P. C. A	40	1	1
5/5	S. P. C. B	36	4	2
5/6	MISC. REQUIREMENTS	38	2	2
5/7	MISC. REQUIREMENTS	39	1	2
5/10	S. P. C. B - COLUMNS, ETC.	38	2	2
5/13	JOINTS, GLASS MASONRY	40	1	1
5/14	REINFORCEMENT DEVELOPMENT	40	1	1
5/15	CONCENTRATED LOADS	38	3	1
5/16	S. P. C. C - CONSTRUCTION LIMITATIONS	39	1	2
5/17	WALLS, HOLLOW UNIT MASONRY	39	1	2
5/21	SPECIAL INSPECTION STRESSES, CORE TESTS	38	1	3
5/23	MATERIAL LIMITATIONS	35	5	2
5/24	MASONRY WALLS, GROUT AND MORTAR STRENGTH	37	2	3
5/25	MATERIAL LIMITATIONS	38	1	3
5/26	S. P. C. D - CONSTRUCTION LIMITATIONS	38	1	3
5/27	SHEAR WALL AND OTHER REQUIREMENTS	36	3	3

5A/1	CLARIFY DEFINITIONS	40	0	2
5A/3	NEW DEFINITIONS	33	7	2
5A/4	DELETE DEFINITIONS	34	5	3
5A/5	DELETE DEFINITIONS	26	13	3
5A/6	DELETE DEFINITION	37	2	3
5A/7	DELETE DEFINITION	33	7	2
5A/8	MODIFY DEFINITION	39	1	2
5A/9	REFERENCE DOCUMENTS	38	2	2
5A/10	SYMBOLS	38	1	3
5A/11	UNIT CRITERIA AND ABSORPTION	37	2	3
5A/12	GLASS UNITS	34	3	5
5A/13	SHRINKAGE	30	7	5
5A/15	LIME, MORTAR	38	1	3
5A/18	CONSTRUCTION	36	3	3
5A/20	STARTER COURSES	34	5	2
5A/21	CONTACT SURFACES	38	2	2
5A/22	ADJACENT WYTHES	38	2	2
5A/25	UNBURNED CLAY MASONRY	39	1	2
5A/27	TOOTHING	37	3	2
5A/28	MISC. GROUTED MASONRY REQUIREMENTS	39	2	1
5A/30	VERTICAL BARRIERS	36	3	3
5A/32	GROUT THICKNESS	38	2	2
5A/35	GROUTING	38	1	3
5A/38	PARTIALLY REINFORCED MASONRY	37	2	3
5A/39	GLASS MASONRY	29	6	7
5A/40	DETAILED REQUIREMENTS	36	3	3
5A/41	DISSIMILAR UNITS	36	2	4
5A/42	UNREINFORCED MASONRY DESIGN	36	3	3
5A/43	UNREINFORCED SOLID CLAY BRICK MASONRY DSGN. PROC.	36	4	2
5A/45	UNREINFORCED SOLID CLAY MASONRY DESIGN	35	4	3
5A/46	UNREINFORCED CONCRETE MASONRY DESIGN PROCEDURES	35	5	2
5A/48	UNREINFORCED CONCRETE MASONRY DESIGN	36	4	2
5A/51	UNREINFORCED HOLLOW CLAY MASONRY DESIGN	34	7	1
5A/53	UNREINFORCED HOLLOW CLAY MASONRY DESIGN	36	4	2
5A/54	REINFORCED MASONRY	38	2	2
5A/55	ANCHORAGE OF REINFORCEMENT	33	8	1
5A/56	OTHER DESIGN REQUIREMENTS	38	2	2
5A/59	NOMENCLATURE	33	6	3
5A/63	OTHER DESIGN REQUIREMENTS	38	2	2
5A/65	ANCHORAGE REQUIREMENTS	36	5	1
5A/67	OTHER CONSTRUCTION	39	1	2
5A/69	CLARIFICATION OF QUALITY CONTROL	39	1	2
5A/70	SEISMIC QUALITY CONTROL	35	6	1
5A/71	MISCELLANEOUS REQUIREMENTS	38	3	1
5A/73	CORE TESTS AND TABLES	36	3	3
5A/75	TITLE	35	6	1
5A/77	TABLES	36	5	1
5A/79	STRESSES	36	4	2
5A/81	FOOTNOTE	36	4	2
6/1	ALLOWABLE FOR MEMBERS WITH EXTRA CAPACITY	44	3	5
6/2	ALLOWABLE SHEAR	41	5	6
6/3	CATEGORY B MOMENT FRAME	44	3	5
6/4	CATEGORY C AND D MOMENT FRAMES	35	12	5
6/6	BEAM-COLUMN JOINT PANEL ZONE	42	4	6
7/1	REFERENCES FOR PLYWOOD	45	1	6
7/2	REFERENCE FOR ONE AND TWO FAMILY CODE	44	2	6
7/3	PHI FACTOR FOR DIAPHRAGMS AND SHEAR WALLS	39	7	6
7/4	TABLE OF PHI FACTORS	37	8	7
7/5	ECCENTRIC JOINTS	40	5	7
7/7	PLYWOOD SHEAR WALLS IN CATEGORY C	40	6	6
7/8	PLYWOOD SHEAR WALLS IN CATEGORY D	41	4	7
7/10	BOTTOM PLATES	38	8	6
7/11	BLOCKING IN PLYWOOD SHEAR WALLS	34	11	7
7/12	TABLE FOR PLYWOOD SHEAR WALLS	42	4	6
7/13	TABLE FOR PLYWOOD DIAPHRAGMS	41	5	6
7/15	REFERENCES FOR PLYWOOD	44	1	7
8/1	OUT-OF-PLANE BENDING	42	3	7
8/2	COMMENTARY ON OUT-OF-PLANE BENDING	42	2	8
8/3	TABLE 8-B FOOTNOTE ON URBAN AREA	43	2	7
8/4	TABLE 8-B ON VENEERS	44	1	7
8/5	EXCEPTIONS FOR ELEVATORS	37	7	8
8/6	NEW SECTION ON ELEVATORS	37	5	9
8/7	TABLE 8-B ON ELEVATORS	38	5	9
8/8	TABLE 8-C ON ELEVATORS	39	4	9
9/1	TITLE OF CHAPTER 1	42	5	5
9/2	DATE PERMIT ISSUANCE	45	2	5
S/6	ALTERNATE REINFORCEMENT FOR PILES	39	7	6
S/7	EXPOSED STRAND FOR CATEGORY B	34	12	6
S/8	EXPOSED STRAND FOR CATEGORY C	37	9	6

TABLE 2 ITEMS WITH A SIMPLE MAJORITY

ITEM	SHORT TITLE	YES	NO	ABST.
1/2	ACTIVE FAULT DEFINITION	30	19	3
2/22	REVISE TABLE OF DRIFT LIMITS	28	18	6
4/12	USE OF 318 APPENDIX A	28	18	6
4/15	PROVISIONS FOR PRECAST AND/OR PRESTRESSED CONC. ...	24	16	2
5/11	HIGH-LIFT GROUT CONSTRUCTION	22	18	2
5/18	BOLT PLACEMENT	24	17	1
5/20	GROUT	22	19	1
5/28	TABLE 12.1	22	17	3
5A/2	DELETE LOAD-BEARING DEFINITION	25	15	2
5A/17	MIXING	23	16	3
5A/19	JOINTS	23	17	2
5A/23	TEMPLATES	21	20	1
5A/44	UNREINFORCED SOLID CLAY ALLOWABLE STRESSES	21	20	1
5A/50	AXIAL COMPRESSIVE STRESSES FOR WALLS	24	17	1
5A/64	HOOK REQUIREMENTS	21	20	1
5A/68	QUALITY CONTROL	22	19	1
7/9	ANCHOR BOLT SPACING	23	22	7
S/4	SEISMICITY INDEX, MAP AREA 2	25	21	6

	MULTIPLE CHOICE ITEM	A	B	C	D
5/8	MINIMUM WALL THICKNESS: CHOOSE ONE OPTION	16	2	0	14

TABLE 3 ITEMS WITH LESS THAN A SIMPLE MAJORITY

ITEM	SHORT TITLE	YES	NO	ABST.
4/6	CATEGORY D PILE REQUIREMENT	21	24	7
5/9	S. P. C. B - WALLS	16	25	1
5/12	MORTAR AND GROUT STRENGTH	17	24	1
5/19	SHRINKAGE CONTROL	18	23	1
5/22	MATERIAL LIMITATIONS	18	23	1
5A/14	CEMENT	17	23	2
5A/16	GROUT CONSISTANCY	20	20	2
5A/24	TIE PLACEMENT	18	23	1
5A/26	GROUTED MASONRY	19	21	2
5A/29	CLEANOUTS	18	23	1
5A/31	REINFORCED CONSTRUCTION	20	20	2
5A/33	HOLLOW UNIT	14	25	3
5A/34	GROUT PROCEDURES	19	21	2
5A/36	LIFTS	16	23	3
5A/37	CLEANOUT	20	20	2
5A/47	UNREINFORCED CONCRETE MASONRY ALLOWABLE STRESSES ..	18	23	1
5A/49	UNREINFORCED HOLLOW CLAY MASONRY DESIGN PROCEDURES	19	22	1
5A/52	SHEAR WALL STRESSES	19	22	1
5A/57	REINFORCED MASONRY WALLS	19	22	1
5A/58	AXIAL WALL STRESSES	17	23	2
5A/60	SHEAR STRESS ALLOWABLES	20	21	1
5A/61	PLACEMENT	20	21	1
5A/62	AXIAL COLUMN STRESSES	17	22	3
5A/66	WALL SHEAR	19	22	1
5A/72	REQUIRED STRENGTHS	18	22	2
5A/74	MINIMUM THICKNESSES	20	21	1
5A/76	H/T RATIOS	18	23	1
5A/78	STRESSES	16	25	1
5A/80	FOOTNOTE	20	21	1
6/5	AXIAL FORCE IN SPECIAL MOMENT FRAME	19	27	6
7/6	NAILS IN CATEGORY C	21	24	7
7/14	EXCEPTION FOR CONVENTIONAL CONSTRUCTION	20	26	6
S/1	SEISMICITY INDEX, MAP AREA 5	19	27	6
S/2	SEISMICITY INDEX, MAP AREA 4	19	27	6
S/3	SEISMICITY INDEX, MAP AREA 3	21	25	6
S/5	SEISMICITY INDEX, MAP AREA 1	23	23	6

APPENDIX A

RECOMMENDATIONS FOR CHANGE AS BALLOTED BY THE JOINT COMMITTEE

This appendix contains the 198 items balloted by the Joint Committee, arranged as follows:

1/1-1/4	The recommendations of Technical Committee 1 (items 1/3 and 1/4 were replaced on the ballot by items S/4 and S/5).
2/1-2/27	The recommendations of Technical Committee 2.
3/1-3/6	The recommendations of Technical Committee 3 (item 3/4 was replaced on the ballot by item S/6 and item 3/6 was deleted by the Coordinating Committee).
4/1-4/12	The initial recommendations of Technical Committee 4 (items 4/2 and 4/5 were replaced on the ballot by items S/4 and S/7).
4/13-4/19	The supplementary recommendations of Technical Committee 4.
5A/1-5A/81*	The recommendations of Technical Committee 5 for chapter 12A (these are also in the form of corrections marked on the original text of the <u>Tentative Provision</u> and are immediately followed by a tabulation of comments by the committee and by ATC).
5/1-5/28	The recommendations of Technical Committee 5 for chapter 12 (these are in the form of corrections marked on the original text of the <u>Tentative Provisions</u> and are immediately followed by a tabulation of comments by the committee and by ATC).
6/1-6/6	The recommendations of Technical Committee 6.
7/1-7/15	The recommendations of Technical Committee 7.
8/1-8/8	The recommendations of Technical Committee 8.
9/1-9/2	The recommendations of Technical Committee 9.
S/1-S/8	The recommendations of the Coordinating Committee.

* Items 5A/1 - 5A/81 follow items 5/1 - 5/28 in tables 1, 2 and 3 of this report.

COMMITTEE 1

JOINT
BALLOT
NUMBER

2.0 COMMITTEE ACTIONS

2.1 Recommendations for Change

Technical Committee 1 on Seismic Risk Maps recommends to the Joint Committee four changes to Section 1.4 of the ATC Tentative Provisions. The first two originated from within Committee 1.

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

Reason: Contours permit a degree of judgment by the user without penalizing the building industry in portions of the country where earthquake and fault records are not as abundant as in the western states and where the zone boundaries are not well defined. The increase in accelerations between zones is enough to penalize buildings in these border areas, some of which are areas of heavy construction. Contour lines are easier to relocate as additional data become available. Seismic risk in large counties close to zones of active faulting also could be treated more consistently.

Opposing points of view were that code administrators prefer a county-type map, and the specification of zones avoids the need to extrapolate ground acceleration at certain boundaries of the contiguous 48 states.

1/2 (2) Change Section 1.4.4 to read: "No new building or existing building which is, because of change in use, assigned to Category D shall be sited on an active fault." Add the following definition: "An active fault is one on which there is evidence of tectonic movement in the past 10,000 years, i.e., Holocene displacement."

Reason: Editorial improvement, includes definition of an active fault. However, one affirmative vote was cast with the reservation that the definition does not cover fault strands, conjugate faults, or associated faults for which no direct evidence of movement exists but which are so related to an active fault that activity on one may be likely as on the other.

In addition to these proposals, several proposals for changing the values of Seismicity Index in Table 1-B of the ATC Tentative Provisions were sent to Committee 1 for ballot as well as to other Committees. The reasons advanced for the proposed changes are described in detail in Section 4, Correspondence. Briefly, it was felt that buildings located in map areas 1 and 2, and probably map area 3, would remain elastic under design ground accelerations and thus additional requirements for detailing to insure ductility over existing practice are not necessary. The view was also expressed that in map areas 4 and 5, the sudden additional requirements for detailing for ductility are not supported by adequate background studies. The ballot of Committee 1 on the proposed changes to the seismicity index classification resulted in the following proposed changes:

COMMITTEE 2

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-7

ATC-3-06 SECTION REFERENCE: 2.1

Add the following sentence immediately following the definition of 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.

FINAL BALLOT: 5 YES
2 NO
1 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The change introduces consistency with the model building codes and simplifies computations for those locations in which only a small snow load would be considered simultaneously with seismic loads. The minority view was that the current ATC provision (use of from 20% to 70% of the full snow load, depending on the judgement of the building official) was an adequate allowance for the small probability of simultaneous occurrence of snow load and seismic load.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-8

ATC-3-06 SECTION REFERENCE: 2.2

Change the definition to read as follows:

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

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The added phrase clarifies the use of live load reduction based on tributary area when combining the effects of live load and seismic load.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-9

ATC-3-06 SECTION REFERENCE: 2.2

Change the definition of Q_S to read as follows:

- Q_S = The effect of snow load, reduced as permitted in section 2.1.

FINAL BALLOT:
8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The added phrase clarifies the use of the reduction from the full design snow load when combining the effects of snow load and seismic load.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-10

ATC-3-06 SECTION REFERENCE: 3.1

Change the second and third sentences to read as follows:

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. <u>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</u>. Individual members shall be sized. . .

FINAL BALLOT:
8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision permits the use of methods incorporating inelastic models of material behavior, subject to explicit approval of the authority.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-11

ATC-3-06 SECTION REFERENCE: 3.2.1

Change the first subparagraph under soil profile type 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

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The added phrase removes the implication that shear wave velocity tests are necessary in order to class a subsoil material as rock.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-12

ATC-3-06 SECTION REFERENCE: 3.2.3

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.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Two issues are involved in this item. First, the committee decided to retain the provision allowing soil-structure interaction analysis because it felt that Chapter 6, Soil-Structure Interaction, was a worthwhile component of the overall seismic design provisions. Second, the revision to the provision was made in recognition of the possible increase in force effects due to the increased P-delta effect resulting from rotation of the base of a building.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-13

ATC-3-06 SECTION REFERENCE: 3.3.1

Delete the word "Special" from the third sentence of the first paragraph.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Since a "special" inverted pendulum is nowhere distinquished from any other type of inverted pendulum, the removal of the word prevents possible confusion.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-14

ATC-3-06 SECTION REFERENCE: 3.3.2(A)

Change the first paragraph to read as follows:

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.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revisions clarify the original intent of the provision.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-16

ATC-3-06 SECTION REFERENCE: 3.3.4(c)

Change to read as follows:

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.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision has the effect of requiring a check on the ability of the seismic resisting system to maintain vertical load carrying capacity when subject to the lateral displacement of the seismic resisting system in the orthogonal direction. (For example, consider the ability of a bearing and shear wall to support vertical load when the wall is laterally supported by an unbraced frame). The provision already required such a check for structural components that were not part of the seismic resisting system.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-17

ATC-3-06 SECTION REFERENCE: 3.7.2

Change to read as follows:

ORTHOGONAL EFFECTS. In buildings assigned to Category B, the design seismic forces may be applied separately in each of two orthogonal directions.

In buildings assigned to Category 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.

FINAL BALLOT: 7 YES
1 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The objective in revising the provision is to reduce the amount of unnecessary computation required. The committee believes that more improvement toward this objective may be possible and looks for the trial designs to provide such information.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-18

ATC-3-06 SECTION REFERENCE: 3.6.2(A) and 3.7.3

Change the second line of 3.6.2(A) to read as follows: . . .shall conform to the requirements of Sec. 3.7 (except Sec. 3.7.3 and Sec. 3.7.12). . .

Change 3.7.3 to read as follows:

For Buildings assigned to Seismic Performance Categories C or D the design of the building shall consider. . .

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision removes the potential inconsistency of requiring formal consideration of strength discontinuities for buildings in which formal consideration of stiffness discontinuities is not required. Consideration of discontinuities in stiffness need not be formally considered for buildings in Seismic Performance Category B (to wit: Modal Analysis is not required for buildings in Category B with vertical irregularities).

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-19

ATC-3-06 SECTION REFERENCE: 3.7.4

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.

FINAL BALLOT:
- 7 YES
- 1 NO
- 0 ABSTAIN
- 2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

No guidance is given on just how redundancy is to be checked. The revision reduces, slightly, the magnitude of this problem.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-20

ATC-3-06 SECTION REFERENCE: 3.7.5

Change the first line to read:

All parts of the building between separation joints shall be interconnected and the connections shall be. . .

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision clarifies the original intent of the provisions.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-21

ATC-3-06 SECTION REFERENCE: 3.7.9

Change the second line of the third paragraph to read:

. . . .elements of the building attached thereto plus the portion of the seismic shear force at that level, V_x, required to be transferred. . .

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision clarifies the original intent of the provision.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-23

ATC-3-06 SECTION REFERENCE: 4.2

Change the last paragraph to read as follows:

The value of C_s may be determined in accordance with Formula 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 buildings, C_s shall be determined using Formula 4-3 or 4-3a, as appropriate.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision encourages the use of the simple equations for those situations in which the calculation of building period has no impact on the design force level.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 1-24

ATC-3-06 SECTION REFERENCE: 4.5

Delete the last sentence of the last paragraph.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The deleted sentence had potential for creating serious design problems for those buildings using piles or piers as holddown anchors, yet no convincing argument has been forwarded for retaining the deleted sentence.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 1-25

ATC-3-06 SECTION REFERENCE: 2.2 and 4.6.2

Change the definition of P_x to read as follows:

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

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
2 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The original definitions for P_x in sections 2.2 and 4.6.2 were not identical. The revised definition specifies the pertinent load for the investigation of instability

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 2-1

ATC-3-06 SECTION REFERENCE: 2.1

Delete the word "wood" from the definition of SHEAR PANEL.

FINAL BALLOT: 7 YES
0 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

It is possible to design and rely on shear panels constructed from materials other than wood, for example metal studs with gypsum board.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 2-2

ATC-3-06 SECTION REFERENCE: 3.3.4(A)

Revise paragraph 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.

FINAL BALLOT: 7 YES
0 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision explicitly recognizes the improved torsional performance of buildings with the principal seismic resisting elements located on the perimeter by relaxing the requirement for four independent lines of resistance for such buildings.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 2-3

ATC-3-06 SECTION REFERENCE: 3.5

Revise 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.

FINAL BALLOT: 7 YES
0 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision removes any implication that the provisions of chapter 5 constitute the only acceptable procedure for modal analysis and specifically allows the use of site specific design spectra, which is the current state of practice for important buildings in highly seismic areas. The precise limit on base shear given in section 5.8 is easier to understand and apply than the limit on building period given in present wording of section 3.5.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 2-4

ATC-3-06 SECTION REFERENCE: 3.8

Revise by adding the following sentence to the end of the last paragraph of the section:

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.

FINAL BALLOT: 7 YES
0 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This revision is coupled with the revision proposed for table 3-6, (item 2-5 from Committee #2) in which the footnote allowing a higher limit for certain buildings is deleted. The types of buildings described in the revised provision have performed well from a drift standpoint in past earthquakes.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: 2-5

ATC-3-06 SECTION REFERENCE: Table 3-C

Remove the footnote from the table and revise the table to read:

TABLE 3-C
ALLOWABLE STORY DRIFT Δ_a

	Seismic Hazard Exposure Group		
	III	II	I
Δ_a	$0.015h_{sx}$	$0.025h_{sx}$	$0.025h_{sx}$

FINAL BALLOT: 6 YES
1 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The drift limitations of Sec. 3.8 of ATC 3-06 are, for many structures, considerably more restrictive than the UBC and usual current design practice. Because of the lack of a close relationship between story drift and either the amount of inelastic strain or the marnitude of the P-delta problems, and because damage control unrelated to safety is not a code objective, the drift limits have been increased.

The revised values were chosen to minimize the possibility of imposing drift constraints more severe than those reflected in current design practice. In specific instances, however, such as for controlling the magnitude of relative movements at joints, the designer may find it necessary or desirable to impose more restrictive limits.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN   COMMITTEE ITEM NUMBER: 2-6

ATC-3-06 SECTION REFERENCE: 4.6.2

Revise the second sentence of the last paragraph to read as follows:

The design story drift determined in Section 4.6.1 shall be multiplied by the factor $(\frac{0.9}{1-\theta} \geq 1.0)$ to obtain the story drift including P-delta effects.

FINAL BALLOT:
7 YES
0 NO
0 ABSTAIN
3 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision explicitly defines for design use the increase in story drift due to P-delta, and by means of introducing the approximation ($0.9 \simeq 1.0$), it avoids a troublesome discontinuity that would occur when $\Theta = 0.10$.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: C-1

ATC-3-06 SECTION REFERENCE: 3.5, ANALYSIS PROCEDURES

Insert following the fourth prargraph 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 dectility 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 modelling 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 (Model Analysis)."

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: C-2.

ATC-3-06 SECTION REFERENCE: 3.3.1, CLASSIFICATION OF FRAMMING SYSTEMS

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

Secretariat's note: the tables furnished by Henry Degenkolb and distributed to the committee with the minutes of the Phoenix meeting are the starting point for this item. It was my understanding that Rol Sharpe was to produce the finished table.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: C-3

ATC-3-06 SECTION REFERENCE: 4.6, DRIFT DETERMINATION AND P-DELTA EFFECTS

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

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #2, STRUCTURAL DESIGN COMMITTEE ITEM NUMBER: C-4

ATC-3-06 SECTION REFERENCE: 5.5, MODAL BASE SHEAR

A plot should be included in the commentary to illustrate the pattern of spectral coefficients for R and A_v.

Secretariat's note: It is my understanding that Rol Sharpe will provide such a plot.

COMMITTEE 3

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 1e

ATC-3-06 SECTION REFERENCE: 3.2.1

The last paragraph in Section 3.2.1 should be changed to read "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 Profile 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.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Soil Profile Type S_2 is much better than Type S_3. Section 3.2.1 suggests soil profile type S_2 when the soil properties are not known. This did not seem logical. Hence the proposed change was recommended.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS   COMMITTEE ITEM NUMBER: 4e

ATC-3-06 SECTION REFERENCE: 7.2.2

The last sentence in Section 7.2.2 should read "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.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Soils are inherently inelastic materials. To specify stressing below the elastic limit is practically without meaning. Hence, the term "elastic limit" should be replaced with the phrase, "to resist loads at acceptable strains".

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 7e

ATC-3-06 SECTION REFERENCE: 7.5.2

The first sentence in Section 7.5.2 should be changed to read "Individual spread footings unless founded directly on rock, as defined in Section 3.2.1(1), shall be interconnected by ties".

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The comment was made that it is overly conservative to require structural ties between pile caps equal to 25% of the maximum vertical load for a Category B structure. This conservatism is amplified in the commentary of this paragraph where it states, "Lateral soil pressure on pile caps is not a recommended method; and if the soil is soft enough to require ties, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions." There are many cases in which the use of piles is dictated by deep soil deposits and the near surface materials are relatively stiff and strong (such as compact or dense gravels and sands overlying soft clays or controlled, compacted fill over clays or organic soils. In these cases, it would seem reasonable to permit at least a portion of the lateral tie resistance between the pile caps to be provided by lateral soil resistance with some guidance provided. In light of these considerations and after discussing the terminology that would be appropriate, the committee agreed to recommend the change shown above.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 9e

ATC-3-06 SECTION REFERENCE: 7.4.4

At the end of paragraph 2 of Section 7.4.4 (before Item A) the following sentence should be added, "Where special reinforcement at the top of the pile is required alternative measures for containing concrete and maintaining ductility will be permitted provided due consideration is given to forcing the hinge to occur in the contained section.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The committee discussed possible designs for the connection at the top of the pile. It was agreed that the intent was to put the ductile section where the hinge would form. Considering this fact and the comments received, the proposed change was recommended. The minority view as expressed in a comment from Committee 4 to use an exposed strand was rejected by the committee because it was judged that one could not manufacturer a ductile connection between the pile and the pile cap using steel strand. Furthermore, it is at the point where the pile is connected to the pile cap that the greatest damage was observed during the San Fernando and Alaskan earthquakes.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS COMMITTEE ITEM NUMBER: 10e

ATC-3-06 SECTION REFERENCE: 7.5.3(c)

The last sentence in Section 7.5.3(c) should be revised to read, "Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from a dynamic analysis of the soil profile."

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The basis for the proposed change was that prestressed precast concrete piling can withstand considerable curvature and through proper detailing confinement and ductility can be provided.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #3, FOUNDATIONS

COMMITTEE ITEM NUMBER: 12e

ATC-3-06 SECTION REFERENCE: Chapter 6

Chapter 6 should be deleted.

FINAL BALLOT: 4 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

After reviewing Chapter 6 and thoroughly examining the procedures therein, the committee felt strongly that the provisions were not effective in implementing a new concept. Chapter 6 is too complicated for the practicing engineer and it is not justified based on field observations. The sophistication of the analysis is inconsistent with the accuracy of the results and the complexity masks the understanding of the performance of the soil structure system. Further documentation for deletion of Chapter 6 is provided in the minutes for the February 15, 1980 meeting (attachments).

COMMITTEE 4

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete

COMMITTEE ITEM NUMBER: M2

ATC-3-06 SECTION REFERENCE: 1.6.3(A)

Alter the sentence under EXCEPTION to read as follows:

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

FINAL BALLOT:

8 YES

0 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

ACI 318, Appendix A permits ASTM A615 Grades 40 and 60 reinforcement. Mill tests specify actual yield and tensile strengths.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: 81

ATC-3-06 SECTION REFERENCE: Chapter 1, Table 1-B

Assign a Seismicity Index of 1 to Map Area Number 2 and carefully review Map Area Number 3 to determine whether or not certain areas such as New York City should more appropriately be designated as Map Area Number 2 for concrete construction.

FINAL BALLOT: 8 YES

0 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The seismicity indices were introduced as a device to relate the seven map areas (acceleration intensities) with the various levels of detailing requirements, as classified in the four seismic performance categories (A, B, C, and D). The indices and the performance categories have been apparently arbitrarily interrelated with the seismic hazard exposure groups (Table 1-A).

While there is little question about detailing requirements for the highest seismicity (4), and for the lowest seismicity (1), detailing requirements for seismicity index levels of 2 and 3 remain a gray area without adequate background information.

OVER

COMMENT ON PROPOSED CHANGE (continued):

Buildings located in the map areas 1 and 2, subjected to acceleration levels of 0.05, will undoubtedly always remain in the elastic range, requiring no additional ductility details. The acceleration level of 0.10 (map area 3) will, in all probability, create an elastic response in buildings designed in conformity with modern reinforced concrete and steel codes. It should also be considered that current codes (i.e., ACI 318) basically result in ductile members, as provisions over the last 20 years have been devised to eliminate brittleness. To suddenly require additional detailing (also adding 30% of forces in perpendicular direction) in cities like New York and Chicago, based largely on judgment, not necessarily supported by adequate background studies, seems questionable. Seismic code writers bear the responsibility to substantiate the need for any restrictive changes made to codes which have been developed in a consensus process over the last several decades. It is not for industries to prove that such changes are unnecessary and will increase the cost of buildings without adding to their safety.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: M8

ATC-3-06 SECTION REFERENCE: 3.7.12

Delete the third sentence.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Formula 3-2a is "for partial penetration welded steel column splices or for reinforced masonry and other brittle materials, systems, and connections." The implication that prestressed members can have a brittle failure is consistent with the possible behavior of some long span extruded precast prestressed products installed without integral topping. However, where topping, properly reinforced and bonded, is used on such units or the component is a pretensioned or post-tensioned unit including supplementary bonded reinforcement equal to the ACI Code 318-77 specified minimums, such brittle failures do not occur and seismic provisions can be consistent with those for reinforced concrete units.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete

COMMITTEE ITEM NUMBER: B6(1)

ATC-3-06 SECTION REFERENCE: 7.4.4

At the end of the first sentence, second paragraph, add the following sentence:

> The pile cap connection may be made by the use of field-placed dowels anchored in the concrete pile.

FINAL BALLOT:

8 YES
0 NO
0 ABSTAIN
0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This is the presently accepted practice in UBC-79 and CAL-TRANS specifications.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete   COMMITTEE ITEM NUMBER: B6(2)

ATC-3-06 SECTION REFERENCE: 7.4.4(E)

Add the following sentence at the end of paragraph:

The pile cap connection for Category B structures may also be made by developing exposed strand.

FINAL BALLOT: 8 YES

0 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This is the presently accepted practice in UBC-79 and CAL-TRANS specifications.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: M11

ATC-3-06 SECTION REFERENCE: 7.5 and 7.6

Make the following changes:

(a) Delete Section 7.6.

(b) Alter the title of Section 7.5 to read as follows:

SEISMIC PERFORMANCE CATEGORIES C AND D.

(c) Alter the first sentence in Section 7.5 to read as follows:

Buildings classified as Category C or D shall conform to all of the requirements for Category B construction except as modified in this Section.

FINAL BALLOT: 6 YES
2 NO
0 ABSTAIN
0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The use of prestressed concrete piling should not be precluded in seismic categories C and D. Performance requirements should be given for their design. See Committee Item Number M10.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: M10

ATC-3-06 SECTION REFERENCE: 7.5.3

Insert the following in Section 7.5.3:

(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%).

(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%), or a spacing satisfying the following relationship, if it results in a percentage of spiral greater than that given above:

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

OVER

FINAL BALLOT: 7 YES

1 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The use of prestressed concrete piling should not be precluded in seismic categories C and D; performance requirements should be given for their design.

References:

1. Gerwick & Brauner - Design of High-Performance Prestressed Concrete Piles for Dynamic Loading (ASTM STP 670, 1979).

2. Margason - Pile Bending During Earthquakes, lecture series at U.C. Berkeley on Effects of Ground Shaking and Movement on Piles, March 6, 1975.

OVER

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%) for a distance of 2 pile diameter, or 2 times the width of the pile.

COMMENT ON PROPOSED CHANGE (continued):

3. Bertero, Lin, Seed, Gerwick, Brauner, and Fotinos - A Seismic Design of Prestressed Concrete Piling, FIP Congress NYC, May 25, 1974.

4. Margason - Earthquake Effects on Embedded Pile Foundations, paper presented at Pile talk Seminar, San Francisco, March, 1977.

5. Test data from dynamic cyclic prestressed piling tests conducted under the sponsorship of the Prestressed Concrete Manufacturers Association of California.

6. Test data from tests conducted by H. Makita of the Tokyu Concrete Pile Company.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: B7

ATC-3-06 SECTION REFERENCE: 8.2.2

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.

FINAL BALLOT:

8	YES
0	NO
0	ABSTAIN
0	DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

UBC-79: The effect of vertical acceleration should be included in the design of nonstructural components and systems.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: B8

ATC-3-06 SECTION REFERENCE: Chapter 8, Table 8-B

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

FINAL BALLOT: 7 YES

1 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Current practice as outlined in UBC-79.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: M6

ATC-3-06 SECTION REFERENCE: 3.6.3

Alter eighth paragraph, starting with the eighth sentence so as to read:

> The loading is cyclical, so static ultimate load capacities may not be reached. If the combination...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.

FINAL BALLOT: 8 YES

0 NO

0 ABSTAIN

0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Clarification of wording is required to make it consistent with the revised Chapter 11.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED ADDITION TO COMMENTARY

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: M7

ATC-3-06 SECTION REFERENCE: 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.

FINAL BALLOT: 8 YES
0 NO
0 ABSTAIN
0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Considerable simplification that is predictable using beam-analogy concepts (1, 2) and has been proven by testing (2).

1. Hawkins, N.M., Mithcell D. and Symonds, D.W., "Hysteretic Behavior of Concrete Slab to Column Connections," Proc. 6th World Conf. Earthquake Engrg., New Delhi, India, 1977.

2. Hawkins, N.M., "Seismic Response of Concrete Flat Plate Structures," Proc. Seventh World Conference on Earthquake Engrg., Instanbul, 1980.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE ITEM NUMBER: Y1

ATC-3-06 SECTION REFERENCE: Chapter 11 and COMMENTARY

Revise Chapter 11 and Commentary Chapter 11 of ATC 3-06 to read as per 28 May 1980 proposal, as modified in meeting of 4 June 1980, and changes necessary to incorporate those revisions into the remainder of ATC 3-06.

FINAL BALLOT: 7 YES
1 NO
0 ABSTAIN
0 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Chapter 11 is revised to reference the nationally recognized design standard, ANSI/ACI 318-77 "Building Codé Requirements for Reinforced Concrete" for proportioning and detailing concrete structures. Seismic resistance is considered in the overall development of the ACI 318 Standard, including Appendix A on special provisions for earthquake resistance.

Existing Chapter 11 originated from an early draft of a proposal by an ACI 318 Seismic Subcommittee to update the ACI 318 seismic design provisions. The current draft of Appendix A (19 March 1980) now before the Main Committee 318 has undergone numerous revisions. Final Committee action and full ACI consensus balloting is in process.

The revised Chapter 11 is formulated to correlate appropriate ACI 318 design provisions with the four ATC seismic performance categories by reference only without the need for ATC to duplicate the wording already contained in the ACI document.

CHAPTER 11 - Pages 101-110

REVISE CHAPTER 11 TO READ AS FOLLOWS:

CHAPTER 11
REINFORCED CONCRETE

Sec. 11.1 - REFERENCE DOCUMENTS

The quality and testing of concrete and steel materials and the design and construction of reinforced concrete components that resist seismic forces shall conform to the requirements of the references listed in this Section, except as modified by the provisions of this Chapter.

Ref. 11.1 ANSI/ACI 318-77 "Building Code Requirements for Reinforced Concrete" including proposed revision Appendix A* - "Requirements for Reinforced Concrete Building Structures Resisting Forces induced by Earthquake Motions" dated 19 March 1980, American Concrete Institute.

Ref. 11.2 AWS D1.4-79 "Structural Welding Code - Reinforcing Steel" American Welding Society.

Sec. 11.2 - REQUIRED STRENGTH

Required strength to resist seismic forces determined by analysis procedures of Chapter 4 or 5 shall be in accordance with Sec. 3.7.1 in lieu of ACI 318 Section 9.2.3.

Sec. 11.3 - SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any construction permitted by ACI 318, and shall conform to the minimum requirements of ACI 318, excluding Appendix A.

All welding of reinforcement shall conform to Ref. 11.2.

---

* "Appendix A-Requirements for Reinforced Concrete Building Structures Resisting Forces induced by Earthquake Motions," 19 March, 1980; copy attached.

Anchor bolts at tops of columns and similar locations shall be closely enclosed within not less than two #4 or three #3 ties located within 4 inches from top of columns. Allowable loads on anchor bolts shall not exceed those given in Table 11-A.

Sec. 11.4 - SEISMIC PERFORMANCE CATEGORY B

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

11.4.1 - ORDINARY MOMENT FRAMES

Where ordinary moment frames are used for the seismic resisting system, frame components (beams and columns) shall be proportioned to satisfy the additional provisions of ACI 318, Appendix A.3.2, A.3.3, A.4.3, and A.8.2. (See ACI 318 Appendix A.2.1.3).

EXCEPTION:

Where slab systems without beams between supports and supported on columns are used for the seismic resisting system, the following provisions shall apply to slab components in lieu of ACI 318, Appendix A.3.2 and A.3.3.

(A) Area of bottom slab reinforcement not less than $1.3\ V_u / \phi f_y$ shall be provided continuous through or anchored within column supports, where V_u is factored shear force transferred to supporting columns due to gravity loading only. Shear force V_u may be reduced by vertical component of effective prestress force for slab systems with prestressing tendons continuous through or anchored within supporting columns.

(B) In each direction, at least 2 bars shall be provided in both top and bottom of slab and made continuous through or anchored within supporting columns.

(C) At least 60 percent of column strip negative moment reinforcement shall be concentrated between lines that are one and one-half slab thickness (1.5h) outside opposite faces of columns.

(D) Shear strength of slab at slab-column connections shall not be taken greater than $(1 + 4/\beta_c)\sqrt{f'_c}b_o d$ when subject to shear force V_u, where b_o is perimeter of a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than d/2 to perimeter of supporting column.

(E) At discontinuous edges of slabs without an edge beam, reinforcement within a distance 4h on either side of a supporting column shall be detailed to resist torsion at discontinuous edges.

11.4.2 - FRAMING SYSTEMS

All components of the seismic resisting system (moment frames, structural walls, braced frames, and diaphragms) shall be proportioned in accordance with provisions of ACI 318, Appendix A.2.1.

EXCEPTION:

Seismic resisting framing systems not satisfying the requirements of Sec. 11.4.1 may be used if it is demonstrated by experimental evidence and analysis that a proposed system will have strength, stiffness, stability, durability, and energy dissipation capacity equal to or exceeding that provided by a comparable monolithic cast-in-place framing system satisfying Sec. 11.4.1.

Alternatively, seismic resisting framing systems that do not contain required special details or energy dissipating mechanisms may be used if designed for forces determined by the analysis procedures of Chapters 4 or 5 with an R value of 1.5.

Sec. 11.5 - SEISMIC PERFORMANCE CATEGORY C AND D

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

11.5.1 - MATERIAL REQUIREMENTS

Materials used in the components of the seismic resisting system shall conform to ACI 318, Appendix A.2.4 and A.2.5.

11.5.2 - FRAMING SYSTEMS

All components of the seismic resisting system (moment frames, structural walls, braced frames, and diaphragms) shall be proportioned in accordance with provisions of ACI 318, Appendix A.

EXCEPTION:

Seismic resisting framing systems not satisfying the requirements of ACI 318, Appendix A, may be used if it is demonstrated by experimental evidence and analysis that a proposed system will have strength, stiffness, stability, durability, and energy dissipation capacity equal to or exceeding that provided by a comparable monolithic cast-in-place framing system satisfying Appendix A.

Alternatively, seismic resisting framing systems that do not contain required special details or energy dissipating mechanisms may be used if designed for forces determined by the analysis procedures of Chapters 4 or 5 with an R value of 1.5.

11.5.3 - STRUCTURAL DIAPHRAGMS

Cast-in-place topping on precast floor systems may serve as structural diaphragms to transmit inertia forces to seismic resisting elements provided the cast-in-place topping is proportioned and detailed to resist the shear forces under the effects of any loading combination (which could induce tensile or compressive stresses simultaneously to the shear forces). Alternate techniques based on the use of untopped precast and/or prestressed components of concrete floor systems may be used only if shown by test and analysis based on established engineering principles that the floor systems will provide the same strength, stiffness, stability, durability and sufficient energy dissipation capacity as a monolithic cast-in-place ordinary reinforced concrete diaphragm.

11.5.4 - FRAME COMPONENTS NOT PART OF SEISMIC RESISTING SYSTEM

All frame components assumed to be not part of the seismic resisting system shall have demonstrated capabilities satisfying Sec. 3.3.4(c) and shall conform to the requirements of ACI 318, Appendix A.8; except, the lateral deformation requirement of A.8.1 shall not apply. If nonlinear behavior is required in such components to comply with Sec. 3.3.4(c), the critical portions shall be provided with special transverse reinforcement in accordance with ACI 318, Appendix A.3.3 or A.4.4.

11.5.5 - RELATIVE FLEXURAL STRENGTH OF COLUMNS

In lieu of ACI 318, Appendix A.4.2, the following shall apply for relative strength of columns.

At any joint where the framing columns resist a factored axial compressive force larger than ($A_g f'_c/10$), the moment in the plane of the frame considered and about the center of the joint corresponding to the flexural strengths of the columns or column shall exceed that corresponding to the flexural strengths of the beams framing into the joint. If this requirement is not satisfied for certain beam-column connections, the remaining columns in the building frame and connected flexural members shall comply and shall be capable of resisting the entire shear at that level accounting for the altered relative rigidities and torsion resulting from the omission of elastic action of the nonconforming beam-column connections. In addition, the columns framing into the affected joint shall be provided with special lateral reinforcement as specified in ACI 318, Appendix A.4.4 throughout their entire story height. Column flexural strengths shall be calculated for the most critical axial design force consistent with the direction of the seismic forces considered.

TABLE 11-A

ALLOWABLE SHEAR AND TENSION ON BOLTS[1]

DIAMETER (inches)	MINIMUM EMBEDMENT[2] (inches)	SHEAR (lbs)	TENSION (lbs.)
1/4	2½	500	360
3/8	3	1100	900
1/2	4	1900	1700
5/8	5	3000	2700
3/4	5½	4300	4050
7/8	6	5900	5750
1	7	7700	7500

[1]Values shown are for minimum concrete compressive strength of 3000 psi at 28 days.

Values are for natural stone aggregate concrete and bolts of at least A-307 quality. Bolts shall have a standard bolt head or equal deformity in the embedded portion.

Values are based upon a bolt spacing of 12 diameters with a minimum edge distance of 6 diameters. Such spacing and edge distance may be reduced 50 percent with an equal reduction in value. Use linear interpoloation for intermediate spacings and edge margins.

[2]A minimum embedment of 9 bolt diameters shall be provided for anchor bolts located in the top of columns for buildings located in Seismicity Index Areas 3 and 4.

COMMENTARY CHAPTER 11 - Pages 449-459

REVISE COMMENTARY CHAPTER 11 TO READ AS FOLLOWS:

COMMENTARY

CHAPTER 11: REINFORCED CONCRETE

For the proper detailing of reinforced concrete construction for earthquake resistance, design standard ANSI/ACI 318-77 "Building Code Requirements for Reinforced Concrete" is referenced. Seismic resistance is considered in the overall development of the ACI 318 Standard, including an Appendix A on Special Provisions for Reinforced Concrete Building Structures to Resist Forces Induced by Earthquake Motions.

Chapter 11 is formulated to reference appropriate ACI 318 design provisions within the four ATC seismic performance categories (A through D). ACI 318 Appendix A refers to zones of different seismicity (Zones 0 through 4) for application of the special provisions for seismic design. For application of Appendix A within the ATC Seismic performance categories, buildings assigned to ATC Category A are interpreted as located in Zone 0 or 1 (regions of no or minor seismic risk), requiring no special provisions for seismic design. Buildings assigned to ATC Category B are interpreted as located in Zone 2 (regions of moderate seismic risk) per Appendix A.2.1.3. Buildings assigned to ATC Category C and D are interpreted as located in Zones 3 and 4 (regions of high seismic risk), per Appendix A.2.1.4. The proportioning and detailing requirements for frames and walls resisting seismic forces are summarized as follows:

	Category A	Category B	Categories C & D
Frame	ACI 318-77	Appendix A.2.1.3	Appendix A
Wall	ACI 318-77	ACI 318-77	Appendix A

For buildings in seismic performance category A, no special provisions are required; the general requirements of ACI 318-77 apply for proportioning and detailing concrete structures.

The code sections cited in ACI 318, Appendix A.2.1.3 for ordinary moment frames (beam-column framing systems) in seismic performance Category B govern reinforcement details of the beam and column components as follows:

	Beams	Columns
Longitudinal reinforcement	A.3.2	A.4.3
Transverse reinforcement	A.3.3	A.8.2

For slab systems without beams between column supports, the slab components of the frame are detailed in accordance with the special EXCEPTION provisions of Sec. 11.4.1.

There are no special requirements for other structural or nonstructural components of buildings in Category B.

In regions of high seismic risk (Categories C and D), the entire building, including the foundation and nonstructural elements, must satisfy ACI 318 Appendix A.

It should be noted that a structural system in a higher category (D being higher than A) must satisfy the requirements specified for the lower categories: A structural frame which forms part of the seismic resisting system of a Category C building must satisfy all of the frame requirements of ACI 318 Appendix A, including Appendix A.2.1.3.

Sec. 11.2 - REQUIRED STRENGTH

Calculations to determine the strength of structural components and members are to be based on Ref. 11.1; except, the factored loads and load combinations to resist seismic forces must be in accordance with Sec. 3.7.1 in lieu of ACI 318 Section 9.2.3. This exception is necessary so that the required strength for seismic resistance, Sec. 3.7.1, is compatible with the design forces specified in Chapter 3.

Sec. 11.3 - SEISMIC PERFORMANCE CATEGORY A

Construction qualifying under Category A as identified in Table 1-A (Chapter 1) may be built with no special detail requirements for earthquake resistance except for ties around anchor bolts as indicated in Sec. 11.3. "Closely enclosed" is intended to mean that the ties should be located within 3 to 4 bolt diameters of the bolts.

Sec. 11.4 - SEISMIC PERFORMANCE CATEGORY B

A frame used as part of the lateral force resisting system in Category B as identified in Table 3-B is required to have certain details which are intended to help sustain integrity of the frame when subjected to deformation reversals into the nonlinear range of response.

For beam and column framing systems, the reinforcement details of ACI 318 Appendix A.3.2 and A.3.3 apply for beam components and A.4.3 and A.8.2 apply for column components.

For slab and column framing systems, the slab component must satisfy the special EXCEPTION provisions of Sec. 11.4.1, in lieu of A.3.2 and A.3.3. Columns must satisfy the provisions of A.4.3 and A.8.2. For slab-column connections, paragraph (A) provides slab reinforcement through a column to support the slab gravity load in the unexpected event that a punching failure occurs. Paragraph B) specifies a minimum amount for that reinforcement. Concentration of negative moment reinforcement at the column as provided by paragraph (C), is required to create a situation whereby the total negative moment reinforcement across the entire slab width will yield simultaneously. Without the heavier concentration of reinforcement, the slab region at the column will yield considerably before the outer regions of the slab, with markedly decreased lateral load stiffness. Paragraph (D) in effect limits the shear stress caused by gravity loads to a sufficiently low value so that the slab-column connection will have a ductility ratio of at least 2. Paragraph (E) ensures that if shear or torsional cracks develop at the slab edges, properly detailed reinforcement is present to control cracking.

As shown in Fig. A there should be top and bottom bars in the slab paralleling and as close to the discontinuous edge as possible, continuous through the column and enclosed within transverse reinforcement having a spacing not greater than 0.5d. The transverse reinforcement can be closed hoops, hairpin stirrups projecting ℓ_{as} beyond the face of the column as shown in Fig. A or slab bars bent to satisfy the requirements for hairpin.

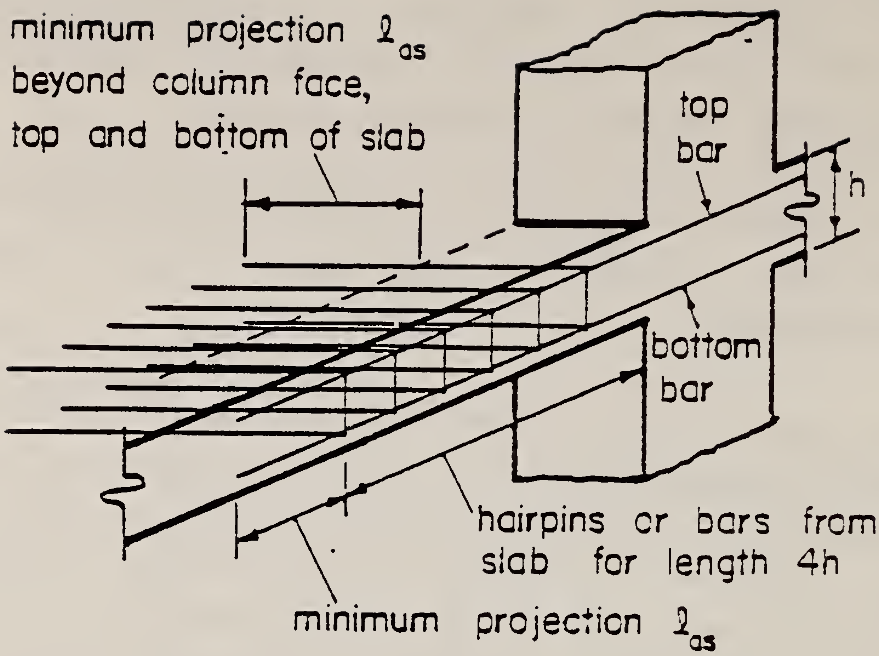


Fig. A - Reinforcement Details
Satisfying Section 11.4.1 (E)

Structural (shear) walls of buildings in Category B are to be built in accordance with the general requirements of ACI 318-77.

Sec. 11.5 - SEISMIC PERFORMANCE CATEGORY C AND D
In regions of high seismic risk, the entire building, including the foundation and nonstructural elements, must satisfy all of the requirements of ACI 318 Appendix A.

Appendix A contains special proportioning and reinforcement detailing requirements which are currently considered to be the minimum for producing a monolithic reinforced concrete structure with adequate proportions and details to make it possible for the structure to undergo a series of oscillations into the inelastic range of response without critical decay in strength. The demand for integrity of the structure in the inelastic range of response is consistent with the rationalization of design forces specified in Chapter 3.

Field and laboratory experience which has led to the special proportioning and detailing requirements in ACI 318 Appendix A has been predominantly with monolithic reinforced concrete building structures. Therefore, the projection of these requirements to other types of reinforced concrete structures, which may differ in concept or fabrication from monolithic construction, must be tempered by relevant physical evidence and analysis. Precast and/or prestressed elements may be used for earthquake resistance provided it is shown that the resulting structure will satisfy the safety and serviceability (during and after the earthquake) levels provided by monolithic construction.

A detailed explanation of the specific provisions of ACI 318 Appendix A is contained in the ACI Code Commentary to Appendix A.

11.5.2 - FRAMING SYSTEMS

The strength and "toughness" requirements for framing systems not satisfying the requirements of ACI 318 Appendix A refer to the concern for the integrity of the entire lateral-force structure at lateral displacements anticipated for ground motions corresponding to design intensity. Depending on the energy-dissipation characteristics of the structural system used, such displacements may have to be more than those for a monolithic reinforced concrete structure.

For systems that remain elastic or that have limited special details for energy dissipation, such as assemblages of precast and/or prestressed concrete, appropriate R-factors should be used to reflect damping characteristics and energy dissipation. For example, $R \simeq 1\frac{1}{2}$ can be used for systems responding primarily elastically to account for damping, and $R \simeq$ up to $2\frac{1}{2}$ may be used for walls with properly distributed web reinforcement that will assure good distribution of cracks and thus provide a degree of energy dissipation.

11.5.4 - FRAME COMPONENTS NOT PART OF SEISMIC RESISTING SYSTEM

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 ACI 318 Appendix A.8. 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.

Slab systems without beams between supports (flat plates) of normal proportions and detailed as specified in Sec. 11.4.1 (EXCEPTION) will not undergo any significant yield until story drifts greater than those allowable. (Table 3-C).

. OTHER REVISIONS TO INCORPORATE NEW CHAPTER 11 - (REINFORCED CONCRETE) INTO ATC 3-06

1. SEC. 1.6.3(B) - PAGE 32
   Change reference "ACI 318-71" to "ACI 318-77"

2. SEC. 2.1 DEFINITIONS - PAGE 37
   Revise the following definitions:

   CROSS-TIE is a continuous bar, No. 3 or larger in size, having a 135-degree hook with a ten-diameter extension at one end and a 90-degree hook with a six-diameter extension at the other end. The hooks shall engage hoop bars and be secured to longitudinal bars.

   HOOP is a closed tie or continuously wound tie (not smaller than No. 3 in size) the ends of which have 135-degree hooks with ten-diameter extensions, that encloses the longitudinal reinforcement.

   JOINT, LATERALLY CONFINED is a joint where members frame into all four sides of the joint and where each member width is at least three-fourths the column width.

   In definition of BRACED FRAME, add the following sentence at the end: "In Chapter 11, reinforced concrete braced frames may be referred to as structural trusses."

   In definition of ORDINARY MOMENT FRAME change reference "Sec. 11.6" to "Sec. 11.4.1."

   In definition of SPECIAL MOMENT FRAME change reference "Sec. 11.7" to Sec. 11.5."

   Add the following definitions:

BOUNDARY ELEMENTS are portions along the edges of walls and diaphragms strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms may also have to be provided with boundary elements.

COLLECTOR ELEMENTS are elements which serve to transmit the inertial forces within the diaphragms to elements of the lateral-force resisting systems.

3. SEC. 2.2 SYMBOLS - PAGE 40
   Delete symbols A_{ch}, A_{sh}, f_{yh}, h_c, P_n, s_h
   Add the following new symbols and definitions:

   b_o = perimeter of critical section for slabs, Sec. 11.4.1
   d = distance from extreme compression fiber to centroid of tension reinforcement, Sec. 11.4.1
   f'_c = specified compressive strength of concrete, psi
   f_y = specified yield strength of reinforcement, psi
   h = overall thickness of member, Sec. 11.4.1
   V_u = factored shear force due to gravity loading, Sec. 11.4.1.

4. TABLE 3-B - PAGE 52
   Revise footnote (4) to read as follows:

   [4]As defined in Sec. 11.5

5. SEC. 7.5.3(C) - PAGE 75
   Change reference "Sec. 11.6.2" to "Ref. 11.1, ACI 318 Appendix A.8.2"

6. SEC. 12.5.1(D) - PAGE 114
   Change paragraph (1) to read as follows:

   "1. Ref. 11.1, ACI 318 Appendix A.5.3 when of reinforced concrete or Chapter 10 when of structural steel."

APPENDIX A - REQUIREMENTS FOR REINFORCED CONCRETE BUILDING STRUCTURES RESISTING FORCES INDUCED BY EARTHQUAKE MOTIONS

A.0-Notation

A_c = net area of concrete section resisting shear, bounded by web thickness and section height, sq.in.

A_{ch} = cross-sectional area of a structural element measured out-to-out of transverse reinforcement, sq.in.

A_{cp} = area of concrete section resisting shear of an individual pier, sq.in.

A_g = gross area of section, sq.in.

A_{sh} = total cross-sectional area of transverse reinforcement (including cross-ties) within spacing "s" and perpendicular to dimension "h"

A_v = total cross-sectional area of shear reinforcement within spacing "s" and perpendicular to longitudinal axis of structural element, sq.in.

A_{vf} = total cross-sectional area of reinforcement perpendicular to a construction joint, sq.in.

b = effective compressive flange width of a structural element, in.

f'_c = specified compressive strength of concrete, psi

f_y = specified yield stress of reinforcement, psi

f_{yh} = specified yield stress of transverse reinforcement, psi

h" = cross-sectional dimension of column core measured c-to-c of confining reinforcement

ℓ_{ah} = anchorage length for a bar with a standard hook as defined in Section A.1

ℓ_{as} = anchorage length for a straight bar

ℓ_o = minimum length, measured from joint face along axis of structural element, over which transverse reinforcement must be provided, in.

P_j = minimum factored compressive force at a construction joint (positive for compression), lb.

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural element, in.

s_o = maximum spacing of transverse reinforcement, in.

V_j = nominal shear force at a construction joint, lb.

γ = dimensionless factor reflecting the influence of confinement of a joint by structural elements framing into joint

ρ = reinforcement ratio, ratio of nonprestressed tension reinforcement

= area at a section to the product "bd"

ρ_a = A_{sa}/A_c; where A_{sa} is the projection on A_c of total area of reinforcement crossing the plane of A_c

ρ_b = reinforcement ratio on a plane perpendicular to A

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out)

ϕ = strength reduction factor

A.1-Definitions

Cross-Tie - A continuous bar, No. 3 or larger in size, having a 135-degree hook with a ten-diameter extension at one end and a 90-degree hook with a six-diameter extension at the other end. The hooks shall engage hoop bars and be secured to longitudinal bars.

Hoop - A closed tie or continuously wound tie (not smaller than No. 3 in size) the ends of which have 135-degree hooks with ten-diameter extensions, that encloses the longitudinal reinforcement.

Structural Walls - Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions.

Structural Diaphragms - Structural elements, such as floor and roof slabs, which transmit the inertial forces to the lateral-force resisting elements.

Structural Trusses - Assemblages of reinforced concrete elements subjected primarily to axial forces.

Lateral-Force Resisting System - That portion of the structure composed of elements proportioned to resist forces related to earthquake effects.

Base of Structure - The level at which the earthquake motions are assumed to be imparted to the building.

Boundary Elements - Portions along the edges of walls and diaphragms strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms may also have to be provided with boundary elements.

Collector Elements - Elements which serve to transmit the inertial forces within the diaphragms to elements of the lateral-force resisting systems.

Anchorage Length for a Bar with a Standard Hook - The shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent, perpendicular to the axis of the bar anchored, to the outer edge of the hook.

Lightweight Concrete - Concrete in which any part or all of the aggregates has been replaced by lightweight material.

Shell Concrete - Concrete outside the transverse reinforcement confining the concrete.

A.2-General Requirements

A.2.1-Scope

A.2.1.1-Appendix A contains special requirements for design and construction of reinforced concrete elements of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

A.2.1.2-The provisions of Chapters 1 through 17 shall apply except as modified by the provisions of Appendix A.

A.2.1.3-In regions of moderate seismic risk*, reinforced concrete frames resisting forces induced by earthquake motions shall be proportioned to satisfy, in addition to the requirements of Chapters 1 through 17, only Sections A.3.2, A.3.3, A.4.3, and A.8.2 of Appendix A.

A.2.1.4-In regions of high seismic risk**, all components of reinforced concrete structures shall satisfy all requirements of Appendix A.

A.2.1.5-A reinforced concrete structural system not satisfying the requirements of Appendix A may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying Appendix A.

A.2.2-Analysis and proportioning of structural elements

A.2.2.1-The interaction of all structural and nonstructural elements which materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

A.2.2.2-Rigid elements assumed not to be a part of the lateral force resisting system may be used provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural elements which are not a part of the lateral-force resisting system shall also be considered.

A.2.2.3-Structural elements below the base of structure required to transmit forces resulting from lateral loads to the foundation shall also comply with the requirements of Appendix A.

A.2.2.4-All structural elements assumed not to be part of the lateral force resisting system shall conform to Section A.8.

*Regions falling in Zone 2 as defined by the Uniform Building Code

**Regions falling in Zones 3 and 4 as defined by the Uniform Building Code

A.2.2.5-Except as required otherwise in Appendix A, structural elements and connections shall be proportioned to resist the load effects with adequate strength in accordance with the provisions of this code using the load factors and strength reduction factors specified in Chapter 9.

A.2.3-Strength reduction factors

Strength reduction factors shall be as given in Chapter 9 except for the following:

A.2.3.1-The strength reduction factor shall be 0.6 for any structural element if its nominal shear strength is less than the shear corresponding to its nominal flexural strength for the design loading conditions.

A.2.3.2-The strength reduction factor for axial compressive force shall be 0.5 for all frame elements with factored axial compressive forces exceeding ($A_g f'_c/10$) if the transverse reinforcement does not conform to Section A.4.

A.2.3.3-Strength reduction factor for anchorage length of reinforcement shall be 0.65.

A.2.4-Concrete in elements resisting earthquake-induced forces

The specified 28-day compressive strength, f'_c, of the concrete shall be not less than 3,000 psi. The specified 28-day compressive strength, f'_c, shall not exceed 4,000 psi for lightweight concrete.

A.2.5-Reinforcement in elements resisting earthquake-induced forces

A.2.5.1-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.

A.2.5.2-Splices in the reinforcement effected through welding or mechanical connections shall conform to Sections 12.15.3.1 through 12.15.3.4.

A.3-Flexural elements of frames

A.3.1-Scope

The requirements of this section apply to frame elements (a) resisting earthquake-induced forces (b) proportioned primarily to resist flexure, and (c) satisfying the following conditions:

A.3.1.1-Factored axial compressive force on the element shall not exceed ($A_g f'_c/10$).

A.3.1.2-Clear span for the element shall not be less than four times its effective depth.

A.3.1.3-The width-to-depth ratio shall not be less than 0.3.

A.3.1.4-The width shall not be less than 10 in. or more than the width of the supporting element (measured on a plane perpendicular to the longitudinal axis of the flexural element) plus distances on each side of the supporting element not exceeding three-fourths of the depth of the flexural element.

A.3.2-Longitudinal reinforcement

A.3.2.1-At any section of a member subjected to bending, the reinforcement ratio, ρ, for the top and for the bottom reinforcement, shall not be less than ($200/f_y$) and shall not exceed 0.025 at any section. At least two bars shall be provided continuously both top and bottom.

A.3.2.2-The positive-moment strength at the face of the joint shall be not less than one-half of the negative-moment strength provided at that face of the joint. The negative- and the positive-moment strengths at any section along the length of the element shall not be less than one-fourth the maximum moment strength provided at the face of either joint.

A.3.2.3-Lap splicing of flexural reinforcement is permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement over the lap length shall not exceed d/4 or 4 in. Lap splices shall not be used (a) within the joints, (b) within a distance of twice the member depth and the face of the joint, and (c) at locations where analysis indicates flexural yielding in connectin with inelastic lateral displacements of the frame.

A.3.2.4-Welded splices and mechanical connections conforming to Sections 12.15.3.1 through 12.15.3.4 may be used for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is 24 in. or more, measured along the longitudinal axis of the frame element.

A.3.3-Transverse reinforcement

A.3.3.1-Hoops shall be provided in the following regions of frame elements:

(1) Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member.

(2) Over lengths equal to twice the member depth on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

(3) Wherever compression reinforcement is required by analysis.

A.3.3.2-The first hoop shall be located nor more than 2 in. from the face of a supporting member. Maximum spacing of the hoops shall not exceed (a) d/4, (b) eight times the diameter of the smallest longitudinal bars, (c) 24 times the diameter of the hoop bars, and (d) 12 in.

A.3.3.3-Where hoops are required, longitudinal bars shall have lateral support conforming to Section 7.10.5.3.

A.3.3.4-Where hoops are not required, stirrups shall be spaced at no more than d/2 throughout the length of the member.

A.3.3.5-Hoops in flexural elements may be made up of two pieces of reinforcement: a stirrup having 135-degree hooks with ten-diameter extensions anchored in the confined core and a cross-tie to make a closed hoop. Consecutive cross-ties shall have their 90-degree hooks at opposite sides of the flexural element.

A.4-Frame elements subjected to bending and axial load

A.4.1-Scope

The requirements of this section apply to frame elements (a) resisting earthquake-induced forces (b) having a factored axial compressive force exceeding ($A_g f'_c/10$) and (c) satisfying the following conditions:

A.4.1.1-The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in.

A.4.1.2-The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

A.4.2-Relative Strength of Columns

A.4.2.1-At any joint where the framing columns resist a factored axial compressive force larger than ($A_g f'_c/10$), the sum of the flexural strengths of the columns calculated for the maximum design axial force shall exceed the sum of the flexural strengths of the beams framing into that joint in the same vertical plane. The flexural strengths shall be summed such that the column moments oppose the beam moments, and the check shall be made in both directions.

A.4.2.2-If Section A.4.2.1 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in Section A.4.4 over their full height if the factored axial force in those columns, related to earthquake effect, exceeds $(A_g f'_c/10)$.

A.4.3-Longitudinal reinforcement

A.4.3.1-The reinforcement ratio, ρ, shall not be less than 0.01 and shall not exceed 0.06.

A.4.3.2-Lap splices are permitted only within the center half of the member span. Welded splices and mechanical connections conforming to Sections 12.15.3.1 through 12.15.3.4 may be used for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 24 in. or more, along the longitudinal axis of the reinforcement.

A.4.4-Transverse reinforcement

A.4.4.1-Transverse reinforcement as specified below shall be provided unless a larger amount is required to resist shear by Section A.7.

(1) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s, shall not be less than that indicated by Eq. (A-1).

$$\rho_s = 0.12\, f'_c/f_{yh} \qquad \text{(A-1)}$$

and shall not be less than that required by Eq. (10-5).

(2) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by Eq. (A-2) and (A-3).

$$A_{sh} = 0.3\ (sh''f'_c/f_{yh})\left[(A_g/A_c) - 1\right] \qquad \text{(A-2)}$$

$$A_{sh} = 0.12\ (sh''f'_c/f_{yh}) \qquad \text{(A-3)}$$

(3) Transverse reinforcement may be provided by single or overlapping hoops. Cross-ties of the same size and spacing as the hoops may be used. Each end of the cross-tie shall engage a peripheral longitudinal

reinforcing bar. Consecutive cross-ties shall be alternated end-for-end along the longitudinal reinforcement.

(4) If the core of the member is sufficient to resist the forces resulting from the specified combination of dead load, live load, and earthquake effects, compliance with Eq. (A-2) and (10-5) is not required.

A.4.4.2-Transverse reinforcement shall be spaced at distances not exceeding (a) one-quarter of the minimum member dimension and (b) 4 in.

A.4.4.3-Cross-ties or legs of overlapping hoops shall not be spaced more than 14 in. on center in the direction perpendicular to the longitudinal axis of the structural element.

A.4.4.4-Transverse reinforcement in amount specified in Section A.4.4.1 through A.4.4.3 shall be provided over a length from each joint face and on both sides of any section where flexural yielding may occur in connection with inelastic lateral displacements of the frame. The length shall not be less than (a) the depth of the member at the joint face or at the section where flexural yielding may occur, (b) one-sixth of the clear span of the member, and (c) 18 in.

A.4.4.5-Columns supporting reactions from discontinued stiff elements, such as walls or trusses, shall be provided with transverse reinforcement as specified above over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds $(A_g f'_c/10)$.

A.5-Structural Walls, diaphragms, and trusses

A.5.1-Scope.

The requirements of this section apply to structural walls and trusses serving as parts of the earthquake-force resisting systems as well as to diaphragms, struts, ties, chords and collector elements which transmit axial forces induced by earthquake. Frame elements, resisting earthquake forces, not complying with Section A.3 or A.4, shall comply with this section.

A.5.2-Reinforcement

A.5.2.1-The reinforcement ratio, ρ, for structural walls shall not be less than 0.0025 along the longitudinal and transverse axes. Reinforcement spacing each way shall not exceed 18 in. The reinforcement required by analysis for shear strength shall be distributed uniformly.

A.5.2.2-At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_c\sqrt{f'_c}$.

A.5.2.3-Structural-truss elements and elements of structural diaphragms having compressive stresses exceeding 0.2 f'_c, shall have special transverse reinforcement, as specified in Section A.4.4, over the total length of the element. The special transverse reinforcement may be discontinued at a section where the calculated compressive stress is less than 0.15 f'_c. Stresses shall be calculated for the factored forces using a linearly elastic model of the element considered.

A.5.2.4-All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, chords, and collector elements shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section A.6.4.

A.5.3-Vertical boundary members for structural walls

A.5.3.1-Boundary members shall be provided at edges of structural walls for which the maximum extreme-fiber stress, corresponding to factored forces including earthquake effect, exceeds 0.2 f'_c unless the entire wall element is reinforced to satisfy Section A.4.4. The boundary member may be discontinued at a level where the calculated compressive stress is less than 0.15 f'_c.

A.5.3.2-Boundary members shall have transverse reinforcement as specified in Section A.4.4 along their full length.

A.5.3.3-Boundary members and similar elements shall be designed to carry all gravity loads on the wall, including tributary loads and self-weight, as well as the vertical force required to resist the overturning moment caused by earthquake.

A.5.3.4-Transverse reinforcement in the walls shall be anchored within the confined core of the boundary member to develop the yield stress in tension of the transverse reinforcement.

A.6-Joints of frames

A.6.1-General requirements

A.6.1.1-Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is 1.25 f_y.

A.6.1.2-Strength of joint shall be governed by the appropriate strength reduction factors specified in Section 9.3. Section A.2.3.1 shall not apply to joints.

A.6.1.3-Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to Section A.6.4 and in compression according to Chapter 12.

A.6.2-Transverse reinforcement

A.6.2.1-Transverse hoop reinforcement, as specified in Section A.4.4 shall be provided within the joint, unless the joint is confined by structural elements as specified in Section A.6.2.2.

A.6.2.2-Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by Section A.4.4 shall be provided where members frame into all four sides of the joint and were each member width is at least three-fourths the column width.

A.6.2.3-Transverse reinforcement as required by Section A.4.4 shall be provided through the joint to provide confinement for longitudinal reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

A.6.3-Shear stress

A.6.3.1-The design shear strength of the joint shall not exceed $\gamma A_j \sqrt{f'_c}$ for normalweight concrete. The coefficient γ shall not exceed 16 if members frame into all vertical faces of the joint and if each framing member covers at least three-quarters of the width and three-quarters of the depth of each joint face. Otherwise, the coefficient γ shall not exceed 12.

A.6.3.2-For lightweight concrete, the joint shear stress shall not exceed three-quarters of the limits given in Section A.6.3.1, where A_j is the minimum sectional area of the joint in a plane parallel to the axis of the reinforcement generating the design shear commentary force.

A.6.4-Anchorage length for reinforcement in tension

A.6.4.1-The anchorage length, ℓ_{ah}, for a bar with a standard 90-degree hook in normalweight concrete shall not be less than $8d_b$, 6 in., and the length required by Eq. (A-4).

$$\ell_{ah} = f_y d_b / 100\phi \sqrt{f'_c} \tag{A-4}$$

for bar sizes No. 3 through No. 11.

For lightweight concrete, the anchorage length for a bar with a standard hook shall not be less than $10d_b$,7.5 in., and 1.25 required by Eq. (A-4).

A.6.4.2-The 90-degree hook shall be located within the confined core of a column or of a boundary member.

A.6.4.3-For bar sizes No. 3 through No. 11, the anchorage length, ℓ_{as}, for a straight bar shall not be less than (a) twice the length required by Section A.6.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in. and (b) 2.8 times the length required by Section A.6.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in.

A.6.4.4-For bar sizes No. 14 and No. 18, the anchorage length for a straight bar shall not be less than 1.5 times that required by Section A.6.4.3.

A.6.4.5-Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary member.

A.7 Shear-strength requirements

A.7.1-Design forces

A.7.1.1-For frame elements subjected primarily to bending, the design shear force shall be determined from consideration of the statical forces on the portion of the element between faces of the joints. It shall be assumed that moments of opposite sign, corresponding to probable strength, act at the joint faces and that the member is loaded with the factored tributary gravity load along its span. The moments corresponding to probable strength shall be calculated using the properties of the member at the joint faces without strength reduction factors and assuming that the stress in the tensile reinforcement is equal to at least 1.25 f_y.

A.7.1.2-For frame elements subjected to combined bending and axial load, the design shear shall be determined from consideration of the forces on the member, with the nominal moment strengths calculated for the maximum factored axial compressive design force on the column, acting at the faces of the joints.

A.7.1.3-For structural walls, diaphragms and trusses, the design shear force shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Section 9.2.

A.7.2-Transverse reinforcement in frame elements

A.7.2.1-For determining the required transverse reinforcement in frame elements in which the earthquake-induced shear force determined in accordance with Section A.7.1.1 represents one-half or more of total design shear, the quantity V_c shall be assumed to be zero if the factored axial compressive force, related to earthquake effects, is less than $(A_g f'_c/20)$.

A.7.2.2-Stirrups or ties required to resist shear shall be hoops over lengths of members as specified in Sections A.3.3, A.4.4, and A.6.2.

A.7.3-Shear strength of structural walls and diaphragms

A.7.3.1-The nominal shear strength, V_n, of structural walls and diaphragms shall not exceed that given by Eq. (A-5).

$$V_n = A_c(2\sqrt{f'_c} + \rho_a f_y) \qquad \text{(A-5)}$$

A_c = net area of concrete section resisting shear bounded by web thickness and height of section.

ρ_a = reinforcement ratio A_{sa}/A_c, where A_{sa} is the projection on A_c of total area of reinforcement crossing the plane of A_c.

f'_c = compressive strength of the concrete in psi.

f_y = yield strength of reinforcement perpendicular to the area A_c.

A.7.3.2-Reinforcement ratio ρ_b, indicating the amount of reinforcement perpendicular to the direction of reinforcement corresponding to ρ_a, shall be equal to or exceed ρ_a.

A.7.3.3-The nominal shear strength of all wall piers sharing a common lateral force shall not exceed $8A_c\sqrt{f'_c}$ where A_c is the total cross-sectional area and the nominal shear strength of any one of the individual wall piers shall not exceed $10\ A_{cp}\sqrt{f'_c}$ where A_{cp} represents the sectional area of the pier considered.

A.7.3.4-The nominal shear strength of horizontal wall elements shall not exceed $10\ A\sqrt{f'_c}$ where A represents the sectional area of a horizontal wall element.

A.8-Frame elements not proportioned to resist forces induced by earthquake motions.

A.8.1-All frame elements assumed not to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load carrying capacity with the structure assumed to have deformed laterally four times that calculated for the specified lateral forces. Such elements shall satisfy the minimum reinforcement requirements specified in Sections A.3.2.1 and A.5.2.1 as well as those specified in Chapters 7, 10, and 11.

A.8.2-All frame elements with factored axial compressive forces exceeding $(A_g f'_c/10)$ shall satisfy the following special requirements unless they comply with Section A.4.4.

A.8.2.1-Ties shall have 135-degree hooks with extensions not less than six tie diameters or 4 in. Cross-ties, as defined in this Appendix, may be used.

A.8.2.2-The maximum tie spacing shall be s_o over a length ℓ_o measured from the joint face. The spacing s_o shall be not more than (a) eight diameters of the smallest longitudinal bar enclosed, (b) 24 tie diameters, and (3) one-half the least cross-sectional dimension of the column. The length ℓ_o shall not be less than (a) one-sixth of the clear height of the column, (b) the maximum cross-sectional dimension of the column, and (c) 18 in.

A.8.2.3-The first tie shall be within a distance equal to 0.5 s_o from the face of the joint.

A.8.2.4-The tie spacing shall not exceed 2 s_o in any part of the column.

A.9-Construction joints

A.9.1-Construction joints in structural walls, diaphragms, and other members resisting lateral forces induced by earthquake shall be designed to resist the design forces at the joint.

A.9.2-Where shear is resisted at a construction joint solely by friction between two roughened concrete surfaces and dowel action, the factored shear force across the joint shall not exceed V_j determined from Eq. (A-6).

$$V_j = A_{vf}f_y + 0.75\ P_j \tag{A-6}$$

where A_{vf} represents the total amount of reinforcement (including flexural reinforcement) normal to the construction joint acting as shear-friction reinforcement and P_j is the algebraic sum of the gravity and earthquake forces on the joint surface acting simultaneously with the shear. For lightweight concrete, the shear strength V_j calculated from Eq. (A-6) shall be multiplied by 0.75.

A.9.3-The surfaces of all construction joints in elements resisting lateral forces shall be thoroughly roughened.

APPENDIX A - REQUIREMENTS FOR REINFORCED CONCRETE BUILDING STRUCTURES RESISTING FORCES INDUCED BY EARTHQUAKE MOTIONS

A.2-General requirements

.A.2.1-Scope

This chapter contains a set of specifications which are currently considered to be the minimum requirements for producing a monolithic reinforced concrete structure with adequate proportions and details to make it possible for the structure to undergo a series of oscillations into the inelastic range of response without critical decay in strength. The demand for integrity of the structure in the inelastic range of response is created by the rationalization of design forces specified by documents such as the 1974 report of the Seismology Committee of the Structural Engineers Association of California.[A.1]

The lateral design forces specified in Reference A.1 are considerably less than those corresponding to linear response for the anticipated earthquake intensity.[A.2, A.3, A.4] As a properly detailed reinforced concrete structure responds to strong ground motion, its effective stiffness decreases and its capability to dissipate energy increases. These developments tend to reduce the response accelerations or lateral inertia forces with respect to those forces calculated for a linearly elastic model of the uncracked and moderately damped structure.[A.5] Thus, the use of design forces representing earthquake effects such as those in Reference A.1 requires that the structure be able to respond in the inelastic range without critical failures. The extent of required nonlinear response is not explicitly established. It is a function of the type and strength of the structure as well as the nature of the ground motion. It is generally assumed that, with the currently used design forces and anticipated earthquake motions, the rotations at connections of reinforced concrete frames are likely to exceed six times the yield rotation. A structural wall similarly proportioned, would be likely to develop relatively less inelastic response. In either case it is essential to have a lateral-force resisting system which will sustain a substantial portion of its strength as it is subjected to successive reversals of displacements into the inelastic range.

The perennial question of a trade-off between strength and special detail requirements has been considered at length. Given a design earthquake intensity or a design response spectrum indexed by an effective peak acceleration, it appears plausible to soften or relinquish some of the detail requirements if the design strength is increased with respect to the minimum code requirement. However, available knowledge on ground motion and structural response to such motion does not make precise estimates of inelastic displacement possible for all structures at large. Furthermore, it is not currently possible to devise explicit quantitative relationships between the required extent and number of inelastic displacements and required reinforcing details. The choice is between (1) a system with sufficient strength to respond to the ground motion within the linear or nearly linear range of reponse and (2) a system with special details to permit nonlinear response without critical loss of strength. The requirements in this appendix have been developed in relation to the second option, on the assumption that the design forces are based on Reference A.1 or a comparable document having a similar approach to the determination of design forces.

The code sections cited in Section A.2.1.3 (which refers to zones of moderate seismic risk) govern reinforcement details of the structural-frame components as follows:

	Girders	Columns
Longitudinal Reinforcement	A.3.2	A.4.3
Transverse Reinforcement	A.3.3	A.8.2

Requirements of Section A.8.2, which have been developed for columns not resisting earthquake effects in high seismic risk zones, apply to columns designed for earthquake effects in moderate seismic risk zones.

There are no special requirements for other structural or nonstructural components of buildings in zones of moderate seismic risk.

In regions of high seismic risk, the entire building, including the foundation and nonstructural elements, must satisfy Appendix A (Section A.2.1.4).

Field and laboratory experience which has led to the special proportioning and detailing requirements in Appendix A has been predominantly with monolithic reinforced concrete building structures. Therefore, the projection of these requirements to other types of reinforced concrete structures, which may differ in concept or fabrication from monolithic construction, must be tempered by relevant physical evidence and analysis. Precast and/or prestressed elements may be used for earthquake resistance provided it is shown that the resulting structure will satisfy the safety and serviceability (during and after the earthquake) levels provided by monolithic construction.

The "toughness" requirement in Section A.2.1.5 refers to the concern for the integrity of the entire lateral-force structure at lateral displacements anticipated for gound motions corresponding to design intensity. Depending on the energy-dissipation characteristics of the structural system used, such displacements may have to be more than those for a monolithic reinforced concrete structure.

A.2.2-Analysis and proportioning of structural elements

It is assumed that the distribution of strength to the various components of a lateral-force resisting system will be guided by the analysis of a linearly elastic model of the system acted on by the factored forces.

Because the design basis is assumed to admit nonlinear response, it is necessary to investigate the stability of the lateral load resisting system and its interactin with other structural and nonstructural elements at displacements larger than those resulting from linear analysis. To handle this problem without having to resort to nonlinear response analysis, one option is to increase by a factor of four the displacements from linear analysis for the specified lateral forces, providing an approximate measure of displacement in the event of a design earthquake, unless the governing code specifies the factors to be used as in References A.6 and A.7.

The main concern of Appendix A is the safety of the structure. The intent of Sections A.2.2.1 and A.2.2.2 is to draw attention to the influence of nonstructural elements on structural response and to hazards from falling objects.

Section A.2.2.3 is included because the base of the structure as defined in analysis may not correspond to the foundation level.

A.2.3-Strength reduction factors.

Section A.2.3.1 refers to brittle elements carrying earthquake induced forces such as low-rise walls or portions of walls between openings of which proportions are such that it becomes impractical to reinforce them to have their nominal shear strength in excess of the shear corresponding to nominal flexural strength for the pertinent loading conditions. This requirement does not apply to the design of connections.

Section A.2.3.2 is included to discourage the use of tied columns to resist earthquake induced forces.

The strength reduction factor of 0.65 is to be used in Eq. (A-4) in determining anchorage length of reinforcing bars with standard hooks. It applies only to anchorage of reinforcement essential to the integrity of the lateral-force resisting structure.

A.2.4-Concrete in elements resisting earthquake-induced forces

The requirements of this section refer to the concrete quality in frames, trusses, or walls proportioned to resist earthquake-induced forces. The maximum design compressive strength of lightweight-aggregate concrete is limited to 4,000 psi primarily because of paucity of experimental and field data on the behavior of elements, made with lightweight concrete, subjected to displacement reversals in the nonlinear range.

A.2.5-Reinforcement in elements resisting earthquake-induced forces

The use of longitudinal reinforcement with substantially higher strength than assumed in design may lead to primary shear or bond failures which are to be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, an upper limit is placed on the strength of the steel.

To insure adequate inelastic rotation in frame elements it is essential to use a reinforcement with an ultimate stress well in excess of the yield stress. For the same reason, any splice must be able to develop a stress equal to 1.25 times the nominal yield stress of the reinforcement.

A.3-Flexural elements of frames

A.3.1-Scope

This section refers to horizontal elements of girders of frames resisting lateral loads induced by earthquake motions. If any horizontal element is subjected to an axial design compressice force exceeding $(A_g f'_c/10)$, in addition to the flexure at any section, it is to be treated as a column as described in Section A.4.

Experimental evidence[A.8] indicates that under reversals of displacement into the nonlinear range, behavior of continuous elements having length-to-depth ratios of less than four is significantly different from the behavior of relatively slender elements. Design rules derived from experience with relatively slender elements do not apply directly to elements with length-to-depth ratios less than four, especially with respect to shear strength.

The geometric constraints indicated in Sections A.3.1.3 and A.3.1.4 derive from practice with reinforced concrete frames resisting earthquake induced forces.[A.1]

A.3.2-Longitudinal reinforcement

Section 10.3.3 limits the tensile reinforcement ratio in a flexural member as a fraction of the amount which would produce balanced strain conditions. For a section subjected to bending only and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to "balanced" failure. The same behavioral model (because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete) fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Thus, there is little rationale for continuing to refer to "balanced conditions" in earthquake resistant design of reinforced concrete structures.

The limit of 0.025 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in girders of typical proportions. The minimum requirement of two No. 5 bars, top and bottom, refers again to construction rather than behavioral requirements.

Lap splices of reinforcement (Section A.3.2.3) are prohibited at regions where flexural yielding is anticipated because such splices are not considered reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location are mandatory because of the likelihood of loss of shell concrete.

A.3.3-Transverse reinforcement

Special transverse reinforcement is required primarily for confining the concrete and maintaining lateral support for the reinforcing bars in regions where yielding is expected. Examples of hoops suitable for flexural elements of frames are shown in Figs. A-1 and A-2.

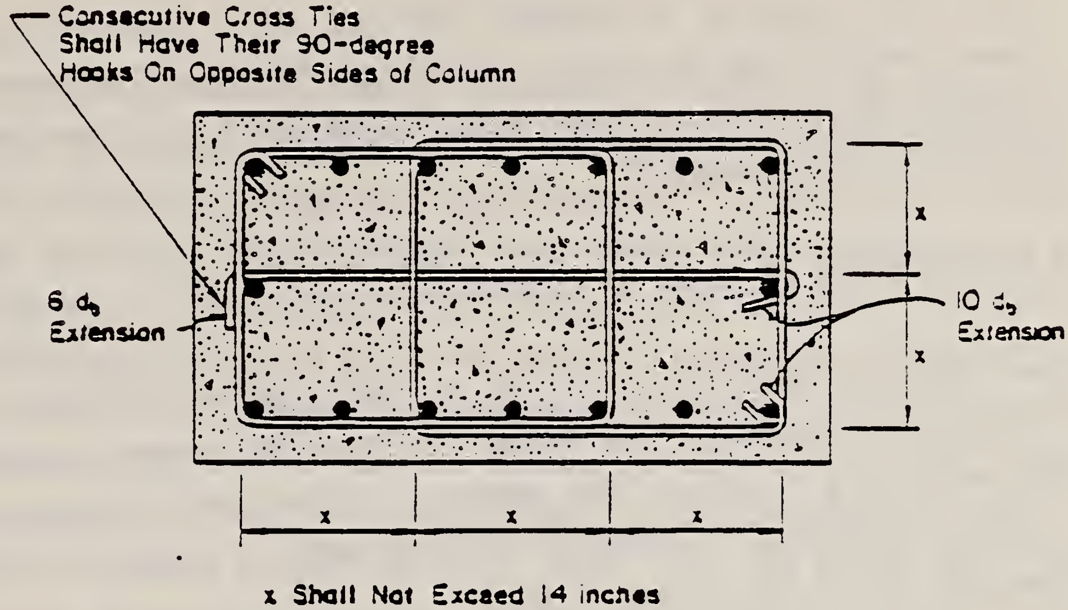


Fig. A-1

In the case of elements with varying strength along the span or elements for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement must be provided throughut the region where yielding is expected.

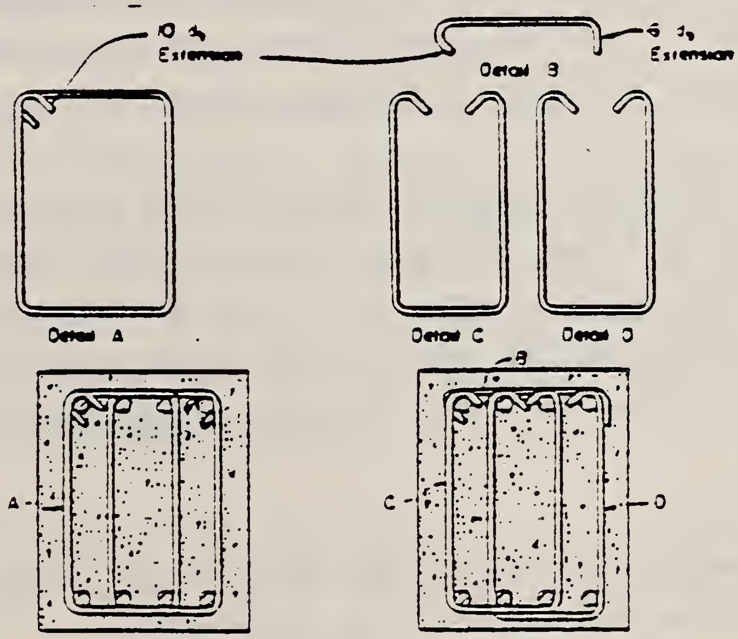


Fig. A-2

A.4-Frame elements subjected to bending and axial load

A.4.1-Scope

This section applies to elements carrying axial loads or columns of frames proportioned to resist earthquake forces. The geometric constraints required by Sections A.4.1.1 and A.4.1.2 follow from previous practice with columns.[A.1]

A.4.2-Relative strength of columns

The intent of Section A.4.2.1 is to limit flexural yielding to the horizontal elements of the frame. If this requirement cannot be satisfied at a joint as, for example, in the case of heavy transfer girders, additional transverse reinforcement is required in the columns affected by forces at the joint.

A.4.3-Longitudinal reinforcement

The lower bound to the reinforcement ratio in elements carrying axial forces as well as flexure refers to the traditional concern for the effects of time-dependent deformations of the concrete as well as desire to avoid a sizeable difference between the cracking and yielding moments. The upper bound reflects concern for steel congestion, load transfer in low-rise construction, and the development of large shear stresses in columns of ordinary proportions.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in those locations quite vulnerable. If lap splices are to be used at all, they must be located near the mid-height where stress reversal is likely to be limited to a smaller stress range than at locations near the joints.

Welding and mechanical splices may occur at any level but not more than half the bars may be spliced at any one section.

A.4.4-Transverse reinforcement

The main reason for the requirements in this section is concern for confining the concrete and providing lateral support to the reinforcement.

For axially compressed elements subjected to steadily increasing load, the effect of helical (spiral) reinforcement on the strength of the confined concrete has been well established.[A.8] Eq. (10-3) follows from the arbitrary design concept that, under axial loading, the maximum capacity of the column before loss of shell be equal to that at large compressive strains with the spiral reinforcement stressed to its useful limit. The toughness of the axially loaded "spiral" column is not directly relevant to its role in the earthquake-resistant frame where toughness or ductility is related to its performance under reversals of moment as well as axial load. For the earthquake problem, there is no reason to modify Eq. (10-5) other than adding the varying lower bound given by Eq. (A-1) which governs for larger columns with gross cross-sectional area, A_g, less than approximately 1.26 times the core area, A_c.

A conservative evaluation of the available data[A.9, A.10, A.11] on the effect of rectilinear transverse reinforcement on the behavior of reinforced concrete would suggest that such reinforcement has little influence on strength but improves ductility although not as effectively as spiral reinforcement. Consequently, there is no explicit basis for relating the required amount of rectilinear transverse to spiral transverse reinforcement. However, it is evident that rectilinear transverse reinforcement is less efficient and if it is used there should be more of it to have an effect comparable to that of spiral reinforcement. Thus, Eq. (A-1) and (A-3) compare to Eqs. (10-5) and (A-2), respectively, but Eq. (A-1) and (A-3) require more reinforcement per unit length of column.

The requirement of Eq. (A-2) which governs for large sections is ignored if the design stresses on the gross section are low.

The transverse reinforcement required by Eq. (10.5), (A-1), (A-2), and (A-3) is distributed over regions where inelastic action is considered to be likely (Section A.4.4.4).

Fig. A-1 shows an example of transverse reinforcement provided by two hoops and a cross-tie.

Dynamic response analyses and field observations indicate that columns supporting discontinued stiff elements such as walls or trusses, tend to develop considerable inelastic response. Therefore, it is required that these columns have special transverse reinforcement throughout their length. This rule covers all columns beneath the level at which the stiff element has been discontinued.

A.5-Structural walls, diaphragms, and trusses

A.5.1-Scope

This section contains requirements for the dimensions and details of relatively stiff structural systems including parts of roof and floor systems transmitting inertia forces as well as walls and trusses. Stubby frame elements, which constitute parts of the lateral force resisting system, must also satisfy the requirements of this section.

A.5.2-Reinforcement

Reinforcement minima (Sections A.5.2.1 and A.5.2.3) follow from preceding codes. The uniform-distribution requirement of the shear reinforcement results from the intent to control the width of inclined cracks. The requirement for two layers of reinforcement in walls carrying substantial design shears is based on the observation that, under ordinary construction conditions, the probability of maintaining the location of a single layer of reinforcement near the middle of the wall plane is quite low. Compressive stress calculated for the factored forces acting on a linearly elastic model of the structural element is used as an index value to determine whether confining reinforcement is required. A calculated compressive stress of 0.2 f'_c on an element is assumed to indicate that integrity of the entire structure is dependent on the ability of that element to resist substantial compressive force under severe cyclic loading. Therefore, transverse reinforcement, as specified in Section A.4.4, is required in such elements to provide confinement for the concrete and the compressed reinforcement (Section A.5.2.4). If this requirement should govern in a solid floor slab, it may be satisfied by a boundary member, as defined in Section A.5.3, rather than providing confinement for the entire slab.

Because the actual stresses in longitudinal reinforcing bars of stiff elements may exceed the calculated stresses, it is required (Section A.5.2.5) that all continuous reinforcement be developed fully.

A.5.3-Vertical boundary members for structural walls

A simplified diagram showing the forces on the critical section A-A of a structural wall acted on by permanent loads, W, and the maximum shear and moment induced by earthquake in a given direction are shown in Fig. A-3. Under the given conditions, the compressed flange is required to resist the acting gravity load plus the total tensile force generated in the vertical reinforcement (or the compressive force associated with the bending moment at section A-A). Recognizing that this loading condition may be repeated many times during the strong motion, it becomes essential to confine the concrete in all wall flanges where the compressive forces are likely to be large as indicated by the design compressive stress exceeding 0.2 f'_c (Sections A.5.3.1 and A.5.3.2). The stress is to be calculated for the factored forces on the section assuming linear response of the gross concrete section. The compressive stress of 0.2 f'_c is used as an index value and does not describe the conditions which may arise at the critical section under the influence of the actual inertia forces for the anticipated earthquake intensity.

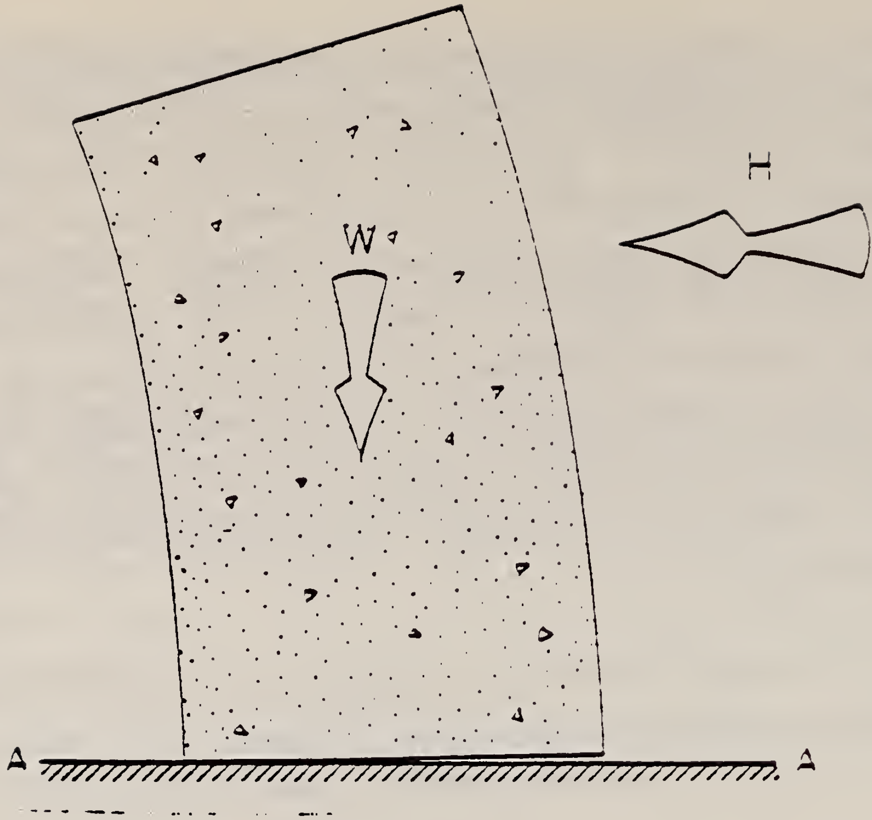


Fig. A-3

The requirement in Section A.5.3.3 is based on the assumption that the boundary element may have to carry all compressive forces at the critical section at the time when maximum lateral forces are acting on the structural wall. The design requirements involve only the section properties: The cross section of the boundary element must have adequate strength (calculated as an axially loaded column) to resist the factored axial compressive force at the critical section.

Because the horizontal reinforcement in walls requiring boundary members is likely to act as web reinforcement, it should be fully anchored in the boundary members which act as flanges (Section A.5.3.4). To achieve this anchorage is made difficult by stress reversals, by and the possibility of large transverse cracks in the boundary members. Wherever feasible standard hooks or mechanical anchorage schemes should be considered.

A.6-Joints of frames

A.6.1-General requirements

Development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with stresses in the flexural reinforcement well in excess of the yield stress. Consequently, joint shear stresses generated by the flexural reinforcement are calculated for 1.25 f_y in the reinforcement (Section A.6.1.1). An explanation of the reasons for the high stresses in girder tensile reinforcement is provided in Reference A.12.

Because the design requirements for joints were developed recognizing that the strength of a joint is typically governed by a brittle mode of failure, Section A.2.3.1 does not apply to joints. The appropriate strength-reduction factor is 0.85 for shear strength.

A.6.2-Transverse reinforcement

However low the calculated shear stresses in a joint of a frame resisting earthquake-induced forces, confining reinforcement (Section A.4.4) must be provided through the joint around the column reinforcement (Section A.6.2.1). Confining reinforcement may be reduced if horizontal members frame into all four sides of the joint as described in Section A.6.2.2.

At joints where the girder is wider than the column, girder reinforcement not passing through the confined core of the column is to be provided with lateral support is provided by framing into the joint.

A.6.3-Shear stress

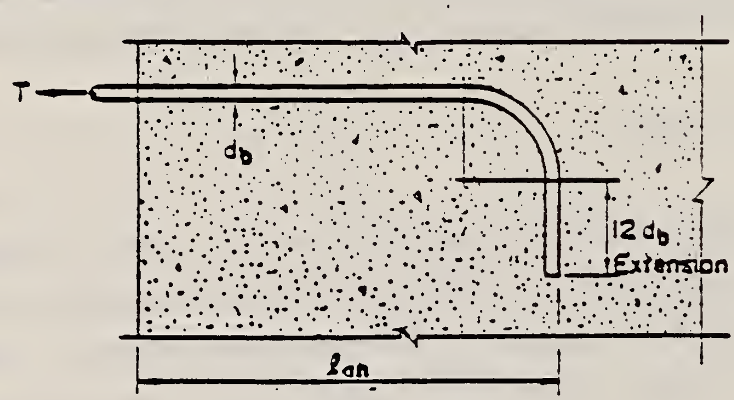
The requirements for the proportioning of joints in Appendix A are based on Reference A.12 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear stress. Because tests of joints[A.19] and deep beams[A.20] indicated that shear strength was not as sensitive,to joint or web reinforcement as implied by the expression developed by ACI Committee 325[A.21] for beams and adopted to apply to joints by ACI Committee 352, it was decided to permit a constant shear stress (derived from the data in Reference A.19) in a joint core having a minimum amount of transverse reinforcement as specified in Section A.6.2.

The designer should note that the joint problem is better solved in proportioning the girders and that tensile stresses may exist in a continuous beam bar through an interior joint at both faces of the joint because of limited anchorage length.

A.6.4-Anchorage length of bars in tension

Eq. (A-4) provides a routine for determining the minimum anchorage length of deformed reinforcing bars with standard hooks embedded in confined concrete made with normalweight aggregate. It is based on recommendations of ACI Committee 408.[A.23] Because the hook is specified to be located in confined concrete, special multipliers for confinement conditions proposed by ACI Committee 408 have been eliminated to simplify calculations.

The anchorage length in tension for a reinforcing bar with a standard hook is defined as the distance, parallel to the bar, from the critical section (where the bar is to be developed) to a tangent drawn to the outside edge of the hook. The tangent is to be drawn perpendicular to the axis of the bar as shown in Fig. A-4.



Note : Hook Must Be
Within Confined Core.

Fig. A-4

For lightweight concrete, the length required by Eq. (A-4) is increased by 25 percent.

Eq. (A-4) is not intended for use with No. 14 and No. 18 bars having standard hooks.

The strength reduction factor to be used in Eq. (A-4) is 0.65 (Section A.2.3.3). It has been reduced from 0.8 proposed by ACI Committee 408 because of the effects of load reversals.

Section A.6.4.3 specifies the minimum anchorage length for straight bars as a multiple of the length indicated by Section A.6.4.1. Case (b) of Section A.6.4.3 refers to "top" bars.

Even though Eq. (A-4) does not apply to hooked No. 14 and No. 18 bars, it is to be used to determine anchorage lengths for straight No. 14 and No. 18 bars. Straight bars are to pass through the confined core in all cases even if the entire anchorage length cannot be accommodated within the confined core.

A.7-Shear-strength requirements

A.7.1-Design forces

In determining the equivalent lateral forces representing earthquake effects for the type of frames considered it is assumed that frame elements will dissipate energy in the nonlinear range of response. Unless a frame element possesses a strength that is a multiple, on the order of three to four, of the design forces, it must be assumed that it will yield in the event of the design earthquake. The design shear force must be a good approximation of the maximum shear that may develop in an element. Therefore the design shear for frame elements is related to the flexural strength of the designed element, rather than to the shear indicated by lateral-load analysis. The conditions described by Sections A.7.1.1 and A.7.1.2 reflect this requirement. Because girders are assumed to develop extensive nonlinear response, design shears in the girders are determined using stresses in the longitudinal reinforcement (1.25 f_y) which reflect the effects of strain hardening.[A.12] Column design shears (Section A.7.1.2) are determined on the basis of limiting moments calculated from interaction diagrams. In both cases strength-reduction factors are assumed to be unity.

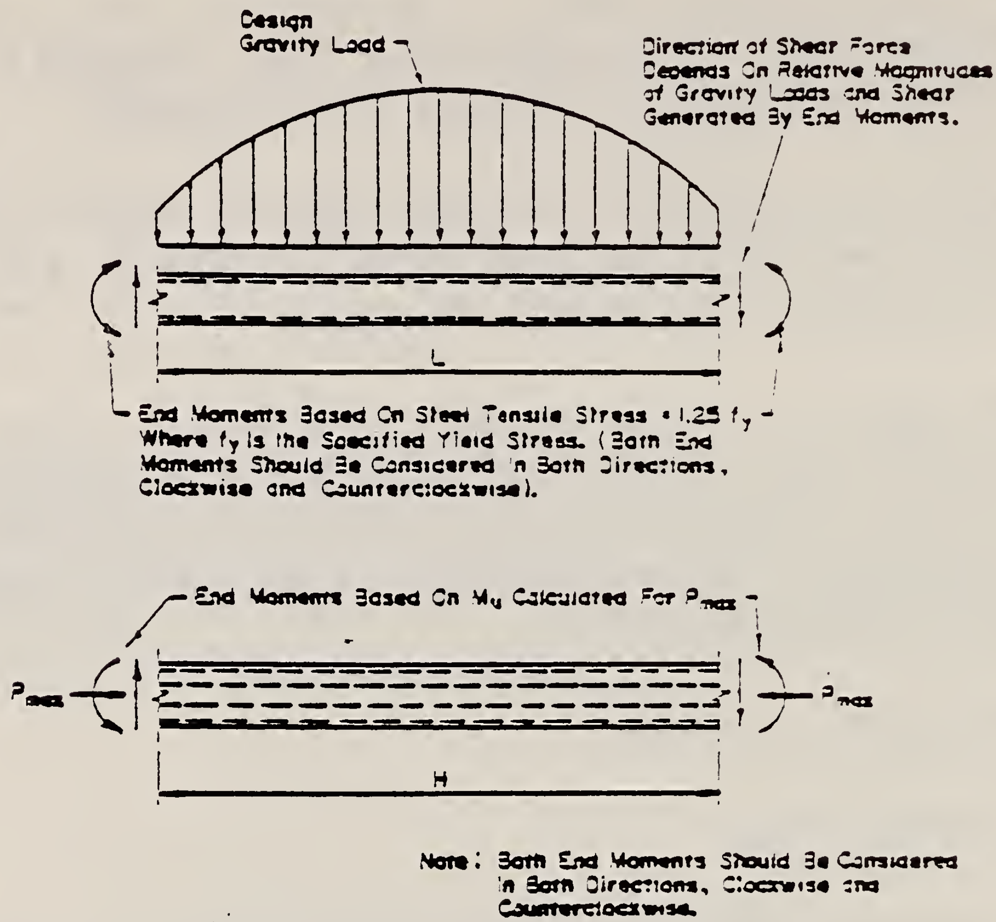


Fig. A-5

Design shears for structural walls, trusses, and diaphragms are obtained from the lateral-load analysis with the appropriate load factors. (However, the designer should consider the possibility of yielding in components of such structures, as in the portion of a wall between two window openings, in which case the actual shear may be well in excess of the shear indicated by lateral-load analysis based on factored design forces.)

The term "probable strength" in Section A.7.1 refers to moment strength calculated with $\phi = 1.0$ and $f_s = 1.25\ f_y$.

A.7.2-Transverse reinforcement in frame elements

Experimental studies at various laboratories of reinforced concrete elements subjected to cyclic loading have demonstrated that more web reinforcement is required to insure a flexural failure if the element is subjected to alternating nonlinear displacements than if the element is loaded in one direction only, the necessary increase of web reinforcement being higher in the case of no axial load. This observation is reflected in the specifications (Section A.7.2.1) by eliminating the term representing the contribution of concrete to shear resistance. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists all the shear with the web reinforcement confining and thus strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not recognize it explicitly.

Because spalling of the concrete shell is anticipated during strong motion, especially at and near regions of flexural yielding, all web reinforcement must be provided in the form of closed hoops as defined in Section A.7.2.2.

A.7.3-Reinforcement in structural walls and diaphragms

Eq. (A-5) has been selected for general use primarily because it provides a simple and familiar vehicle for the determination of the required amount of transverse reinforcement. To differentiate between stubby and slender walls was considered to be unwarranted considering the increased calculation effort the differentiation requires would be likely to offset any economy in material it might effect.

The requirement for the distribution of calculated shear stress in walls working in parallel reflects the need to avoid overloading one of the piers while the others are barely loaded.

"Horizontal wall element" in Section A.7.3.4 refers to wall sections between two vertically aligned openings (Fig. A-6).

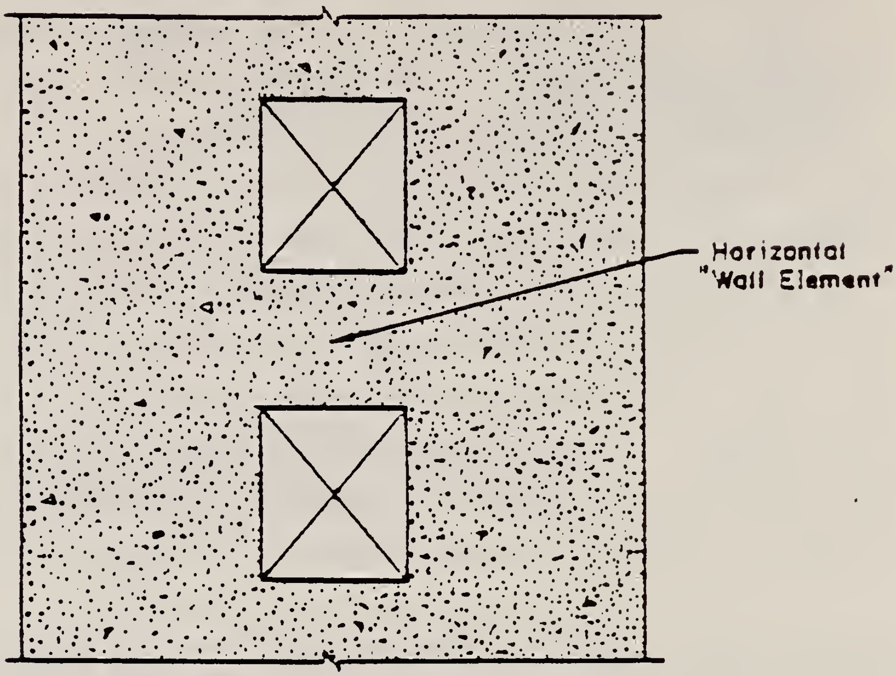


Fig. A-6

A.8-Frame elements not proportioned to resist forces induced by earthquake motions.

The intent of Section A.8.1 is to insure that the parts of the structural system, designed for gravity loading only, will continue to be functional at lateral displacements for which the lateral-force resisting system has been designed. Consequently, the gravity-load system need only accommodate the specified lateral displacements without reduction in gravity-load carrying capacity. Reduction in flexural stiffness of reinforced concrete elements of the gravity-load system may be recognized in calculations. It is not necessary to reinforce the gravity-load system for moments related to lateral forces.

A.9-Construction joints

Construction joints require explicit attention during the design as well as the construction of a building. Eq. (A-6) reflects the influence on shear strength of the estimated net force normal to the construction joint. It should be noted that the normal force related to the lateral motion will reduce the compressive force due to gravity. A positive value for P_n refers to compression on the joint.

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A.19 Meinheit, D. F., and Jirsa, J. O., "The Shear Stregth of Reinforced Concrete Beam-Column Joints," CESRL Report No. 77.1, Univ. of Texas, Austin, Jan. 1977.

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A.22 International Conference of Building Officials, "Uniform Building Code," Whittier, California.

A.23 ACI Committee 408, "Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension, Concrete International, July 1979, pp. 44-46.

JOINT BALLOT NUMBER: 4/13

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: A1

ATC-3-06 SECTION REFERENCE: 11.1

Alter Section 11.1 such that the reference reads as follows:

> "Reference 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."

Final Ballot: 1 Yes

0 No

4 Abstain

3 Did Not Vote

COMMENTS:

This ballot item updates the reference to include the latest version of the ACI Building Code for Concrete (ACI 318-77). The replacement of Section 9.2.3 in the ACI Code by ATC 3-06 Section 3.7.1 reminds the designer that the combination of load effects used in ATC 3-06 is different than that in ACI 318-77.

This ballot item appeared on the first of the two committee letter ballots. The final wording was modified so as to read exactly as revised and approved by the ATC representative. The abstentions were the result of the ballot item being superseded by the committee ballot item Y1 (Joint Ballot Number 4/12). The committee was in full agreement that the reference should be updated, but the issue of adopting Appendix A overshadowed that intent.

JOINT BALLOT NUMBER: 4/14

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: A2

ATC-3-06 SECTION REFERENCE: 11.2

Alter Section 11.2, first paragraph, second sentence by inserting "Precast and/or prestressed" in place of "Precast."

Final Ballot: 5 Yes

0 No

0 Abstain

3 Did Not Vote

COMMENTS:

The intent of the ballot item is to expressly include prestressed concrete as a permissible building material. Initially, the ATC representative was opposed to mention of prestressed construction without any accompanying criteria for its proper design. However, with the introduction of the material contained in committee ballot item M9 (Joint Ballot Number 4/15), the ATC representative approved this change to the existing ATC 3-06 Chapter 11.

JOINT BALLOT NUMBER: 4/15

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: M9

ATC-3-06 SECTION REFERENCE: New Section 11.9

Add the following as a new Section in Chapter 11 immediately following Section 11.8:

Section 11.9 STRUCTURES COMPRISED OF PRECAST AND/OR PRESTRESSED CONCRETE SUBASSEMBLAGES

The provisions of this Section apply to buildings constructed with precast and/or prestressed concrete elements not conforming to the detailing provisions given elsewhere in this Chapter for cast-in-place concrete.

11.9.1 LINEAR ELASTIC DESIGN

Structures with assemblages of precast and/or prestressed concrete components furnishing lateral resistance against seismic forces shall be designed to elastically resist equivalent lateral forces equal to those specified in this document with an R value of 1.0.

OVER

COMMENTS:

The intent of this change to the existing ATC 3-06 Chapter 11 is to provide a clear mechanism by which a designer can use a precast and/or prestressed construction within the framework of the ATC 3-06 provisions. Section 11.9.1 presents a method by which a structure can be designed to resist elastically earthquake forces and which is likely to be an economically viable solution for low-rise construction only (≤ 3 stories). Section 11.2 presents a method which follows the more conventional approach of permitting inelastic action providing the system offers the same behavioral characteristics (e.g. strength, stiffness, damping, etc.) as comparable monolithic cast-in-place ordinarily reinforced concrete construction.

The ATC representative reviewed and approved of the proposed ballot item. There were two reservations of a technical nature expressed by members of the committee. The first concerned the use of an R value of 1.0 in the Linear Elastic Design section. The committee member felt that to be overly conservative and suggested a value of R = 1.5. The other reservation accompanied the "No" vote and was an objection to the lack of a provision limiting the height and/or the number of stories.

11.9.2 "DUCTILE" CONSTRUCTION

Energy dissipating lateral load resisting systems comprised of precast and/or prestressed concrete components shall be permitted provided satisfactory evidence can be shown in the form of experiments, testing, and analysis based upon established engineering principles that the resulting construction complies with the requirements of Sections 3.6 and 3.7 and this Chapter, and that they offer the same strength, stiffness, stability, durability, damping, energy absorption, and energy dissipation capabilities (ductility) as monolithic cast-in-place ordinarily reinforced concrete construction.

Final Ballot:	7	Yes
	1	No
	0	Abstain
	0	Did Not Vote

JOINT BALLOT NUMBER: 4/16

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: M1

ATC-3-06 SECTION REFERENCE: 11.5.1

Alter Section 11.5.1, third paragraph such that it reads 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."

Final Ballot: 8 Yes

0 No

0 Abstain

0 Did Not Vote

COMMENTS:

This change replaces the current wording in ATC 3-06 Chapter 11 with the wording included in the latest draft version of the ACI Committee 318 Appendix A (Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions). The committee was in complete agreement that the Appendix A wording was more desirable than the existing wording. The ATC representative objected to this change because it did not sufficiently emphasize that if ASTM A615 Grade 60 steel is used careful attention must be given to the metallurgy of the steel and the welding practice.

JOINT BALLOT NUMBER: 4/17

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: M3

ATC-3-06 SECTION REFERENCE: 11.8.2

Alter Section 11.8.2 by deleting in its entirety the third paragraph and replace it with the following:

> "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."

Final Ballot: 8 Yes 0 Abstain

0 No 0 Did Not Vote

COMMENTS:

The ballot item modifies the existing complete restriction against the use of untopped precast and/or prestressed components of floor systems as diaphragms. Instead, the change would permit such systems to be considered as diaphragms if it can be shown that the untopped system provides behavior comparable to that of a monolithic cast-in-place ordinarily reinforced concrete diaphragm.

The ballot item was reviewed by the ATC representative who supported its adoption. One committee member, however, expressed reservations about the practicality of verification and the lack of a commentary section giving a clear explanation of the provision's intent.

JOINT BALLOT NUMBER: 4/18

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete COMMITTEE BALLOT NUMBER: M4

ATC-3-06 SECTION REFERENCE: 11.6.1

Four part item

a) Alter Section 11.6.1, second paragraph, second sentence so as to read:

"At least two No. 5 or larger bars shall be provided continuously both top and bottom except in slabs."

b) Alter Section 11.6.1, sixth paragraph, first sentence so as to read:

"Web reinforcement perpendicular to the longitudinal reinforcement shall be provided throughout the length of all members except slabs."

c) Alter Section 11.6.1, seventh paragraph, first sentence so as to read:

"Within a distance equal to twice the effective depth from the end of all members except slabs, the amount...from the end of the member."

OVER

COMMENTS:

The ballot item introduces design provisions for flat slab construction. Such provisions are not present in the existing ATC 3-06 Chapter 11 and it was felt by the committee that such an omission would not be representative of the current building practice in many areas of the nation.

The ATC representative reviewed and approved of the provisions included in this ballot item.

While approving this item, committee members expressed concern about the use of unfactored gravity loads in the proposed equation 11-2. The use of unfactored loads is inconsistent with all other sections of Chapter 11 where factored loads are used.

Four part item (continued)

d) Alter Section 11.6.1 by adding the following paragraph after the seventh paragraph:

"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:

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

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."

Final Ballot: 8 Yes

0 No

0 Abstain

0 Did Not Vote

JOINT BALLOT NUMBER: 4/19

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #4, Concrete   COMMITTEE BALLOT NUMBER: M5

ATC-3-06 SECTION REFERENCE: Commentary C11.5.1

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

> "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)."

Final Ballot: 8 Yes

0 No

0 Abstain

0 Did Not Vote

COMMENTS:

This change to the Commentary emphasizes that flat plate frames are considerably more flexible than other framing systems.

The ATC representative reviewed and approved the proposed ballot item which incorporates his suggested revisions. There was one reservation expressed by a committee member. He felt that while what was stated in the ballot item was true for most "normal proportions" there were exceptions and suggested that the word "will" be replaced by "should."

COMMITTEE 5

CHAPTER 12A

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 Chapters 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. Re-entrant spaces are excluded in the gross area unless these spaces are to be occupied by masonry by portions of adjacent units.

1 (10-0-0) 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-sectional area of a section minus the average area of ungrouted cores or celluar 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.

5A/1 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.

2 (10-0-0) 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.

12A.1.1 Cont.

5A/1 cont. 3 (10-0-0) ~~JOINT, COLLAR. The interior longitudinal vertical joint in a wall between wythes. In grouted masonry construction, it is the grout space.~~

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.

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.

5A/2 4 (10-0-0) ~~LOAD BEARING. Synonymous with Structural.~~

5A/3 5 (8-2-0) 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.

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

5A/4 ~~MASONRY, GROUTED. Construction conforming to Sec. 12A.3.5 is most often referred to as grouted brick construction.~~

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

5A/5 6 (10-0-0) ~~NONBEARING. This term refers to a nonload-bearing component, usually a wall.~~

7 (10-0-0) ~~NONLOAD BEARING. Synonymous with Nonstructural.~~

5A/6 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 structrual components.

8 (9-0-1) ~~PARTIALLY REINFORCED MASONRY. Masonry construction conforming to Sec. 12A.3.7 and other applicable provisions of this Chapter.~~

5A/7 9 (8-2-0) ~~REINFORCED MASONRY. Grouted masonry construction conforming to Sec. 12A.3.5(C) or Hollow Unit Masonry conforming to Sec. 12A.3.6(A). Reinforced masonry shall also conform to other applicable provisions of this Chapter, including Sec. 12A.2.2, 12A.2.4, 12A.6.3 and 12A.6.4.~~

5A/8 REINFORCEMENT RATIO. This is the ratio of the areas of reinforcement to the gross cross-sectional area of the masonry perpendicular to the reinforcement.

10 (10-0-0) SHEAR WALL is a vertical component resisting lateral forces by in-plane shear and flexure. ~~(unless defined elsewhere).~~

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.

5A/9 12A.1.2 REFERENCE DOCUMENTS

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

12A.1.1 Cont.

11 (10-0-0)

MATERIALS AND Testing ~~DESIGN~~	STANDARD DESIGNATION
Building and Facing Brick	
Clay and Shale	ASTM C62, C216, C652*
Sand-Lime	ASTM C73
Method of Test	ASTM C67
Concrete Masonry Units	
Hollow Load-Bearing	ASTM C90
Solid Load-Bearing	ASTM C145
Hollow Nonload-Bearing	ASTM C129
Brick	ASTM C55
Method of Test	ASTM C140
Structural Clay Tile	
12 (9-0-1) For Walls - Load-Bearing	ASTM C34, C212 ,C126
For Walls - Nonbearing	ASTM C56
For Floors	ASTM C57
Cast Stone	ACI 704
5A/9 cont. 13 (9-0-1) **Unburned Clay**	Uniform Building Code Standard 24-~~15~~ 14
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

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

12A.1.2 Cont.

5A/9 cont. 14 (8-0-2)

~~Testing~~

Masonry Assemblies~~, Cores, Mortar and Grout~~ — Sec. 12A.7 & 12A.8

~~Slump Test for Grout~~ — ~~Sec. 12A.8.2~~

~~Rate of Absorption~~ — ~~ASTM C67-73~~

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.

5A/10 15 (9-0-1)

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.

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 = Masonry strength for development length or splice determination, psi. (See Sec. 12A.6.3(D))

f_m = Allowable compressive unit stress, psi.

f'_m = Compressive strength of masonry, psi.

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

12A.1.3 Cont.

f_t = Allowable flexural tensile stress in masonry, psi.

f_v = Allowable unit stress in web reinforcement, psi.

16 (9-0-1) 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 centriod 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.

5A/10 cont. n = Ratio of modulus of elasticity of steel to that of masonry.

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

17 (9-0-1) p = 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.

5A/11 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 Cont.

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.

5A/11 cont. 18 (9-0-1)

12A.1.6 ~~BRICK~~ MASONRY UNIT SURFACES FOR GROUTED MASONRY

~~Masonry~~ units for ~~reinforced and unreinforced~~ grouted masonry shall have all surfaces to which grout is to be applied capable of adhering to grout with sufficient tenacity to resist ~~a~~ the required shearing stress~~. of 100 psi after curing 28 days~~. 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 re-used 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

5A/12 19 (9-0-1)

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 ~~3.5~~ 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 required by this Chapter and Chapter 12 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.

5A/13 20 (9-0-1)

~~12A.1.13 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.~~

12A.1 Cont.

21 (5-3-2) 12A.1.14 CEMENT

Cements for mortar are limited to those allowed by ASTM ~~C270~~ C476, this Chapter and Chapter 12 except masonry cement shall not be used for grout.

5A/14 22 (9-0-1)

~~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 waterproofed 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 be Type I, IA, II, IIA, III, IIIA, or V portland cement, or Type IS, IS-A, IS (MS), IS-A (MS), IP, or IP-A blended hydraulic cement.~~

23 (9-0-1) 12A.1.15 LIME

~~Lime putty shall be made from quicklime or hydrated lime. If made from other than processed pulverized quicklime, the lime shall be slaked and then screened through a No. 16 mesh sieve. After slaking and screening, and before using, it shall be stored and protected for not less than 10 days.~~

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

~~Processed pulverized quicklime shall be slaked for not less than 48 hours and shall be cool when used.~~

5A/15 24 (10-0-0) 12A.1.16 MORTAR

Mortar shall be prepared in accordance with either ~~of~~ the property or proportions procedures given ~~below:~~ in ASTM C270.

- ~~The Property Specifications of ASTM C270 may be used with acceptability based upon the properties of both the ingredients and samples of mortar mixed and tested in the laboratory using the proportions and materials proposed for use. Compressive strengths shall not be less than required by Table 12A-1A.~~

- ~~The Proportion Specifications of ASTM C270 may be used with acceptability based upon the properties of the ingredients, the water retention of laboratory mixed and tested samples, and the proportions of the ingredients summarized in Table 12A-1B.~~

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 Types O and K mortar shall not be used.

~~Masonry units used in foundation walls and footings shall be laid up in Type S or Type M mortar. See Sec. 12A.3 and Chapter 12 for further limitations.~~

12A.1.16 Cont.

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 plasticity, mortar may be retempered with water by the method of forming a basin in the mortar and reworking it. However, any mortar which has ~~become~~
25 ~~harsh shall not be used in the work.~~ hardened or stiffened due to hydration of the
(10-0-0) 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.

5A/15 EXCEPTION:
cont. 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
26 cement, to which may be added not more than 1/10 part hydrated lime ~~or lime putty,~~ and
(9-0-1) 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 ~~or lime putty,~~ 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 Regulator Agency.

27 ~~Coarse grout may be used in grout spaces in grouted masonry 2 inches~~
(10-0-0) ~~or more in width and in grout spaces in filled cell construction having an area of 15 square inches with a least dimension of 2 inches.~~

~~Coarse grout shall be used where the least dimension of the grout space exceeds 5 inches and where otherwise required.~~

(C) CONSISTENCY. Grout shall have a consistency, considering the methods
28 of consolidation to be utilized, to completely fill all spaces to be grouted without segre-
5A/16 (9-1-0) gation.~~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.

(E) MEASURING AND MIXING. Materials for grout shall be measured in suitable
29 calibrated devices. After the addition of water, all materials shall be mixed for at least
5A/17 (9-1-0) three minutes in a ~~drum type batch~~ mechanical 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
5A/18 tests to verify the grout strength. Such tests shall be made in accordance with Sec. 12A.7 and 12A.8.2.

12A.1.17 Cont.

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

30 (10-0-0)

~~EXCEPTION:~~
~~Aluminum equipment may be used if it can be demonstrated that there will be no deleterious effect on the strength of the grout and it is specifically approved by the Regulatory Agency.~~

12A.1.18 REINFORCEMENT

Reinforcement over one-fourth inch (No. 2) in diameter shall be deformed bars.

Sec. 12A.2 <u>CONSTRUCTION</u>

~~At the time of laying all masonry units shall be clean and free of dust. Burned clay and sand-lime units shall be dampened prior to laying with an absorption rate conforming to Sec. 12A.1.6. Surfaces of concrete masonry units to receive mortar shall be dampened by means of a fog spray or equivalent during hot and dry weather, as described in Sec. 12A.2.5. At the time of laying all unburned clay units shall be damp at the surface. All masonry units shall not be so wet that free water is present on the surfaces.~~

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

5A/18 cont. 31 (9-0-1) 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.

~~Surfaces of all masonry units for grouted construction at the time of laying shall be capable of developing the required bond with grout as specified in Sec. 12A.1.6.~~

12A.2.1 JOINTS

32 (9-0-1) All units shall be laid with shoved mortar joints. Solid units shall have all head and bed joints solidly filled. ~~Except for cavity walls, spaces to be grouted, and as provided in Sec. 12A.3.3, all wall joints, collar joints, and joints between wythes shall be solidly filled.~~ unless otherwise approved.

5A/19 33 (9-1-0) All hollow units shall be laid with ~~full~~ 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.~~ unless otherwise specified.

5A/20 34 (8-2-0) ~~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, shall be moistened per the requirements of Sec. 12A.2, 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.~~

5A/21 35 (9-1-0) 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 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.

12A.2.2 Cont.

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.

36 (10-0-0) (A) REQUIREMENTS. Adjacent wythes shall be bonded to each other in accordance with the applicable provisions of ~~Sec. 12A.2.1 and~~ Sec. 12A.3.

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

37 (10-0-0) 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:

~~For unreinforced masonry the mechanical bond shall be provided by one of the following:~~

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

5A/22 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 inch 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.

38 (6-0-4) ~~For stacked bond reinforced grouted or reinforced hollow unit masonry, see Chapter 12. For stacked bond partially reinforced masonry, see Sec. 12A.3.7(A) and Chapter 12. For shear walls see Sec. 12A.6.4 and Chapter 12.~~

12A.2.3 CORBELING

39 (9-0-1) ~~Corbels in unreinforced masonry may be built only into solid masonry walls 12 inches or more in thickness. Corbels in partially reinforced masonry may be built only into masonry walls 12 inches or more in thickness unless the construction provided for the~~

12A.2.3 Cont.

39 cont. ~~corbel is designed and constructed as reinforced masonry. The projection for each course in such corbels and in unreinforced corbels in reinforced masonry construction shall not exceed 1 inch, and the maximum projection shall not exceed 1/2 of the total thickness of the wall when used to support a chimney built into the wall. The top course of all unreinforced corbels shall be a header course.~~

The slope of corbelling (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 one one-half the wall thickness for all other walls.

12A.2.4 REINFORCEMENT

~~Reinforcement shall conform to the requirements of this Section.~~

40 (10-0-0) 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.

41 (10-0-0) (B) SPLICES. Splices in reinforcement may be made only at approved locations or ~~and~~ as indicated on the approved design documents. Splices shall conform to the provisions of Sec. 12A.6.3(D)7.

5A/22 cont. (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.

EXCEPTIONS:

1. Number 11 bars may be used provided the grout cover, measured from 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

42 (9-0-1) ~~No masonry shall be laid when the temperature of the outside air is below 40°F unless approved methods are used during construction to prevent damage to the masonry. Such methods include protection of the masonry for a period of at least 24 hours where Type III portland cement is used in mortar and grout and for a period of at least 48 hours where other cements are used. Materials to be used and materials to be built upon shall be free from ice and snow.~~

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 Cont.

12A.2.6 ANCHORAGE

5A/22 cont. 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 Sec. 3.8.

12A.2.7 BOLT PLACEMENT

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

5A/23 43 (9-1-0) 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 of mortar between 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.

5A/24 44 (7-1-2) 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~~ 4 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

5A/25 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 units. Masonry of unburned clay 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.~~

45 (10-0-0) Unburned clay masonry is that form of construction made with unburned clay stabilized 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.

12A.3 Cont.

12A.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 uniformily distributed shall be provided to the extent of not less than 10 percent of the area of 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

soild concrete or clay masonry units

46 (9-0-1) Solid masonry shall be ~~brick, concrete brick, or solid load bearing concrete masonry units,~~ laid contiguously in mortar.

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

5A/25 cont.

- 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. Unit 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 be not 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.

12A.3.4 Cont.

5A/25 cont. 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 each 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.

47 (10-0-0) 12A.3.5 GROUTED MASONRY -MULTI/WYTHE WALLS

5A/26 48 (4-0-5) ~~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.~~

Grouted multi-wythe is a form of construction in which interior joints between wythes are filled with grout. Only Type M or Type S mortar shall be used.

5A/27 49 (8-1-1) ~~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.~~

50 (10-0-0) ~~Grouting and construction procedures shall conform to the requirements given below.~~

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:

51 (10-0-0) ~~1. All units in the two outer tiers shall be laid with full shoved head and bed mortar joints.~~ Masonry headers shall not project into the grout space.

52 (8-0-2) 1. ~~2.~~ 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 requred to maintain grout thicknesses between masonry units and reinforcement. In members of three or more ~~tiers~~ 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 ~~shall~~ may be used where the grout space exceeds 5 inches in width. ~~The thickness of grout between masonry units and floaters shall be not less than 1 inch.~~ All grout shall be puddled ~~with a grout stick~~ immediately after pouring.

5A/28 53 (10-0-0) 2. ~~3.~~ One exterior ~~tier~~ wythe may be carried up 18 inches before grouting, but the other exterior ~~tier~~ 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. ~~4.~~ If the work is stopped for one hour or longer, the horizontal construction joints shall be formed by stopping all ~~tiers~~ 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:

54 (9-0-1) ~~1. All units in the two tiers shall be laid with full head and bed joints.~~

12A.3.5 Cont.

wythes

5A/28 cont. 55 (10-0-0) 1. ~~a~~. The two ~~tiers~~ 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 ~~tier~~ 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. wythe

5A/29 56 (8-1-1) 2. ~~b~~. Cleanouts shall be provided for each pour by leaving out ~~every other~~ units 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.

57 (10-0-0) 3. ~~c~~. The grout space (longitudinal vertical joint) shall not be less than 2 ~~3~~ inches in width ~~and not~~ nor less than the thickness required by the placement of steel with the required clearances and shall be poured solidly with grout. ~~Masonry walls shall cure at least three days to gain strength before grout is poured.~~

58 (10-0-0) EXCEPTION:
If the grout space contains no horizontal steel, it ~~may be reduced to 2 inches.~~ shall be at least 3 inches.

5A/30 59 (9-0-1) 4. ~~d~~. 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. ~~Unless a true joint occurs at the barrier,~~ 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. ~~e~~. 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. ~~f~~. Grouting shall be done in a continuous pour, in lifts not exceeoing 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.

60 (10-0-0) 7. ~~g~~. Inspection during grouting shall be provideo in accordance with Sec. 12A.7. ~~however, the work shall not qualify for the stresses entitled, "Special Inspection", unless fully inspected per Sec. 1.6.2, 1.6.4, and 12A.7.~~

5A/31 61 (9-1-0) (C) REINFORCEO 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.~~

12A.3.5 Cont.

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.

62 (10-0-0) 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.

5A/32 Where two or more hollow units are used to make up the thickness of an 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.

5A/33 63 (7-3-0) ~~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.~~

GROUTING PROCEDURES

5A/34 64 (8-1-1) (A) ~~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 continuous vertical cell measuring not less than ~~2~~ 1 1/2 inches by 3 inches. If walls are battered or if alignment is offset, the ~~2~~ 1 1/2 inch by 3 inch clear opening shall be maintained as measured from course to course.

65 (10-0-0) Overhanging mortar fins projecting ~~more than the thickness of the mortar joint~~ into the grout space shall be ~~carefully~~ removed. ~~as the work progresses in a manner that prevents the mortar from falling to the bottom of the cells.~~

66 (10-0-0) 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 dimension of the grout space exceeds 5 inches and where otherwise required.

5A/35

67 (10-0-0) 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 ~~units,~~ except as permitted for masonry joint reinforcement. ~~See Sec. 12A.6.3(F) and Chapter 12.~~

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

12A.3.6 Cont.

68 (10-0-0) 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.

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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 two 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:

5A/36 69 (7-2-1) ~~a. Units may be laid up 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.~~

5A/37 70 (8-0-2) b. Cleanouts shall be provided in the foundation or ~~by omitting~~ in the 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.~~ Debris in the grout space shall be 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 constituents 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.

71 (10-0-0) d. Grouting shall be done in a continuous pour and may be done in partial lifts. ~~so as to avoid blowout of units.~~

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.

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72 (10-0-0) e. Inspection during grouting shall be provided in accordance with Sec. 12A.7. ~~however, the work shall not qualify for the stresses entitled, "Special Inspection", unless fully inspected per Sec. 1.6.2, 1.6.3, and 12A.7.~~

73 (9-0-1) ~~12A.3.7 PARTIALLY REINFORCED MASONRY~~

~~Partially reinforced masonry is grouted masonry 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.~~

12A.3.7 Cont.

Partially reinforced masonry shall be designed as unreinforced masonry, except that reinforced masonry areas or elements may be considered as resisting stresses in accordance with the design criteria specified for reinforced masonry provided such elements fully comply with the design and construction requirements for reinforced masonry except as herein noted; however R factors of Table 3-B shall be as required for unreinforced masonry. Only Types M or S mortar shall be used.

(A) REINFORCEMENT. 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 the 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

5A/38 cont. 73 cont. than 8 inches. The vertical reinforcement ratio and the horizontal reinforcement ratio shall each be not less than 0.00027.

Where not prohibited by Chapter 12 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.

Partially reinforced masonry walls shall be considered as reinforced masonry for the purpose of applying Table 12A-2.

12A.3.8 GLASS MASONRY

74 (6-0-4) Masonry of glass blocks may be used in nonloadbearing exterior or interior walls and in openings, either isolated or in continuous bands, provided the glass block panels have a minimum thickness of ~~3.5~~ 3.0 inches at the mortar joint.

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

5A/39 panels shall not exceed 250 square feet of unsupported area nor 25 feet in any dimension.

75 (6-0-4) Glass block shall be laid in Type~~s~~ N ~~M or S~~ mortar by proporation. 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 <u>DETAILED REQUIREMENTS</u>

5A/40 76 (10-0-0) 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

12A.4 Cont.

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.

5A/40 cont. 77 (7-0-1)

Except where specifically allowed otherwise, stresses shall be calculated on actual net dimension 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, this Chapter, or by other governing provisions, ~~specific~~ 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.

12A.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 stresss 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

5A/41 78 (10-0-0)

All masonry walls shall be designed so that allowable stresses are not exceeded ~~and so that their thicknesses are not less than required by the maximum thickness ratios and the minimum thicknesses of Table 12A-2. When a change in minimum thickness requirements occurs between floor levels, the greater thickness shall be carried to the higher floor level. In computing the thickness ratio for cavity walls, the value for thickness shall be determined by Footnote 5 of Table 12A-2. In walls composed of different kinds or classes of units or mortars, the ratio of height to length to thickness shall not exceed that allowed for the weakest of the combination of units and mortars of which the member is composed.~~ and that the provisions of table 12A-2 are satisfied.

79 (10-0-0)

EXCEPTION:
The maximum thickness ratio of Table 12A-2 may be increased ~~and the minimum nominal thicknessess of Table 12A-2 may be decreased when data is submitted which justifies such liberalization and approval is obtained from the Regulatory Agency. For all walls and elements serving to support vertical loads other than induced by the walls or elements themselves such data shall include consideration of the additional eccentricity of vertical load due to deflections perpendicular to the plane of the wall or element and, for unreinforced and partially reinforced masonry, a consideration of stress and stability under reduced vertical loads in accordance with the provisions of Chapters 3 and 4 including Formula 3-2a for unreinforced masonry.~~ when justified by substantiating data.

5A/42 80 (10-0-0)

12A.4.3 PIERS

~~Every structural pier whose width is less than three times its thickness shall be designed and constructed as required for columns.~~

12A.4.3 Cont.

80 cont. ~~Every structural pier in reinforced masonry construction whose width is between 3 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 construction such steel may be in one layer in the form of hairpins.~~

12A.4.4 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.4.5 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.4.6 ARCHES AND LINTELS

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

5A/42 cont.

12A.4.7 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.4.8 END SUPPORT

81 (10-0-0) Beams, girders, or other similar concentrated loads supported by a wall or ~~pier~~ 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 ~~safely~~ distributed to the wall or ~~pier~~ column, or to a continuous reinforced masonry member ~~projecting not less than 3 inches from the face of the wall,~~ or by other approved means.

82 (9-0-1) Joists, precast planks, and similar elements shall have a bearing at least 2.5 inches in length upon solid or grouted masonry elements ~~at least 2.25 inches thick,~~ or other provisions shall be made to distribute the loads ~~safely~~ to the masonry.

Anchorage to the masonry shall conform to Chapter 3.

12A.4.9 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.

12A.4.9 Cont.

~~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.~~

83 (10-0-0) 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 STRENGTHS AND ALLOWABLE STRESSES

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-3 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.8 with tests made both prior to and during construction.

5A/42 cont. 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 12-4 provided other tests are made and certifications are furnished when required by the footnotes to Table 12-4 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 Inspection" unless Special Insoection fully conforming to Sec. 1.6 and 12A.7 is provided.

83A - New (10-0-0) (B) ALLOWABLE STRESSES FOR MASONRY. Except for unreinforced ~~brick~~ masonry designed under the provisions of Sec. 12A.6.2, the allowable stresses for unreinforced masonry are given in Table 12A-3 and for reinforced masonry the allowable stresses are given in Table 12A-5.

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

12A.5.2 Cont.

	POUNDS PER SQUARE INCH
COMPRESSIVE STRESS IN COLUMN VERTICALS:	
40 percent of the minimum yield strength, but not to exceed	24,000
COMPRESSIVE STRESS IN FLEXURAL MEMEBERS:	
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-6. See Sec. 12A.2.7 for construction requirements.

Sec. 12A.6 DESIGN REQUIREMENTS

5A/42 cont.

84 (9-0-1)

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. ~~and the age when the masonry elements may be loaded.~~ All stresses and capacities shall be based on actual net dimensions, thickness and sections.

85 - Mod (10-0-0)

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. ~~An~~ Alternate procedures for design are ~~is~~ 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-3. 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.

12A.6.1(B) Cont.

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 considerations 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.

5?/42 cont.

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 above 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 area of the loaded wythe.

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

12A.6.1(E) Cont.

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.

5A/42 cont. 86 (10-0-0)

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

86A - New (10-0-0)

(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.

5A/43 87 - Mod (8-1-1)

12A.6.2 ALTERNATE DESIGN PROCEDURE FOR UNREINFORCED BRICK MASONRY

For unreinforced brick masonry constructed with only new solid units made from clay or shale conforming to ASTM C62 or ASTM C216 and subject to the limitations of Footnote 4 to Table 12A-4, the alternate design procedure of this Section may be used. The requirements of Sec. 12A.6.1 apply except as specifically modified herein.

The value of the brick masonry design strength, f'_{mb}, for establishing the allowable stresses for use in this Section shall be 0.73 of the value of the masonry compressive strength determined in accordance with Sec. 12A.5.1, i.e.:

$$f'_{mb} = 0.73\ f'_m$$

All plans shall clearly show the values of f'_m, and f'_{mb}, at their required age.

(A) SLENDERNESS RATIOS. The slenderness ratio of a wall shall be taken as the ratio of its effective height, h, to the effective thickness, t, and shall not exceed the smaller of the values determined from Table 12A-2 or as determined by the following formula:

$$\frac{h}{t} \leq 10\ (3 - \frac{e_1}{e_2}) \qquad (12A\text{-}1)$$

where the value of e_1/e_2 is positive where the member is bent in single curvature, and negative where the member is bent in double or reverse curvature. Where e_1 and e_2 are both equal to zero, e_1/e_2 shall be assumed to be zero.

The slenderness ratio of a column shall be the greater value obtained by dividing the effective height h in any direction by the effective thickness t in the corresponding direction and shall not exceed the value determined by the following formula:

$$\frac{h}{t} \leq 5\ (4 - \frac{e_1}{e_2}) \qquad (12A\text{-}2)$$

The minimum thickness and maximum slenderness requirements of Sec. 12A.4.2 shall also be satisfied. However, those requirements and the slenderness limits of the above formulas may be waived in accordance with Sec. 1.5. Conformance to the formulas, by itself, shall not act as a waiver for the requirements of Sec. 12A.4.2. The requirements of the exceptions of that Section, as applicable, shall be satisfied. Where applicable, the design procedures following this Subsection may be used in satisfying the requirements of those exceptions. Particular attention shall be paid to the requirements for stress and stability under reduced vertical load including Formula 3-2a and to transverse loads.

(B) ALLOWABLE VERTICAL LOAD. The allowable vertical loads and bearing stresses shall be determined as follows:

5A/43 cont. | 87 Mod. cont.

12A.6.2(B) Cont.

1. Allowable Vertical Load. The allowable vertical load, P, on an unreinforced masonry wall or column shall be determined in accordance with the following.

a. When e/t or R_e, as applicable, do not exceed 1/3:

$$P = f_m A_g C_s C_e \qquad (12A-3)$$

where f_m is the allowable axial compressive stress from Table 12A-7.

A_g is the cross-sectional area of the element determined from the effective thickness and length.

C_s is the slenderness coefficient as determined by the following formula:

$$C_s = 1.20 - \frac{h/t}{300}\left[5.7 + \left(1.5 + \frac{e_1}{e_2}\right)^2\right] \leq 1.0 \qquad (12A-4)$$

C_e is the eccentricity coefficient, related to the ratio of maximum effective eccentricity to effective thickness, e/t as determined below.

When e/t is equal to or less than 1/20, the value of C_e is 1.0.

When e/t exceeds 1/20 but is equal to or less than 1/6 the value of C_e shall be determined by use of the following formula:

$$C_e = \frac{1.3}{1 + 6\frac{e}{t}} + \frac{1}{2}\left(\frac{e}{t} - \frac{1}{20}\right)\left(1 - \frac{e_1}{e_2}\right) \qquad (12A-5)$$

When e/t exceeds 1/6 but is equal to or less than 1/3, the value of C_e shall be determined by use of the following formula:

$$C_e = 1.95\left(\frac{1}{2} - \frac{e}{t}\right) + \frac{1}{2}\left(\frac{e}{t} - \frac{1}{20}\right)\left(1 - \frac{e_1}{e_2}\right) \qquad (12A-5a)$$

For members subject to tranverse loads greater than 10 pounds per square foot between lateral supports, C_e shall be based on Formula 12A-5 or 12A-5a, whichever is applicable, except e_1/e_2 shall be taken as 1.0.

When the elements are subject to bending about both principal axes, the eccentricity coefficient is related to the ratio:

$$R_e = \frac{e_i t + e_t i}{it} \qquad (12A-6)$$

where e_i is the effective eccentricity about the principal axis which is normal to the length of the element.

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

12A.6.2(B) Cont.

i is the effective length of the element.

t is the effective thickness of the element.

When R_e is equal or less than 1/20 the value of C_e is 1.0.

When R_e exceeds 1/20 but is equal to or less than 1/6, the value of C_e shall be determined by use of Formula 12A-5 except that R_e shall be substituted for e/t.

b. When e/t or R_e, as applicable, exceed 1/3:

For walls and elements subject to bending in one direction only and the ratio e/t exceeds 1/3, the maximum tensile and flexural compression stress in the masonry, assuming linear stress distribution, shall not exceed the values given in Table 12A-7. Where these values are exceeded, the member shall be redesigned and/or reinforced.

For walls and elements subject to bending in both directions, and the ratio R_e exceeds 1/3, the members shall be redesigned and/or reinforced.

See Chapter 12 for modifications under seismic loads.

2. Bearing Stress. The bearing stress under beams, lintels and girders and from similar concentrated loads supported on unreinforced masonry shall not exceed the values set forth in Table 12A-7.

(C) SHEAR WALLS. Design of shear walls shall comply with all applicable provisions of Sec. 12A.6.4 and Chapter 12. In unreinforced shear walls, the effective eccentricity ei about the principal axis which is normal to the length i of the shear wall shall not exceed an amount which will produce tension. In unreinforced shear walls subject to bending about both principal axes, R_e shall not exceed 1/3. Where the effective eccentricity exceeds the values given in this Section, shear walls shall be redesigned or reinforced.

5A/43 cont. | 87 Mod. cont.

Allowable vertical loads on unreinforced shear walls shall be determined in accordance with Sec. 12A.6.2(B) except that the value of h used in determining C_s may be taken as the minimum vertical or horizontal distance between lateral supports.

The allowable shear stresses in unreinforced shear walls shall be taken as the allowable stresses given in Table 12A-7. The allowable shear stress may be increased by 1/5 of the average compressive stress due to dead load at the level being analyzed for all loading combinations except those including seismic loadings. In no case, however, shall the allowable shear stresses exceed the limiting values given in Table 12A-7.

(D) CONSTRUCTION. Masonry designed in accordance with this Section shall have head and bed joints with an average thickness not over 1/2 inch. All interior joints shall be solidly filled.

(Replace Sec. 12A.6.2 with the following:)

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 (BIA-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

5A/43 cont.

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

87 Mod. cont.

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 2/3 of those with special inspection.

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

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d. Allowable shear stresses without special inspection shall be 60% of those with special inspection.

6. Modulii of Elasticity shall conform to Table 12A-5, this chapter.

7. Brick Masonry ultimate compressive strength f'_m for use in ~~the~~ this alternate procedure shall be ~~73% of the values obtained by the provisions of Sec. 12A.5.a(A) of this chapter which otherwise are applicable.~~ taken from Table 12A-4, or shall be 82% of the values obtained by prism testing according to section 12A.8 (h/t =2)

9-0-1

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

5A/45

9. Footnote 4 to Table 12A-4 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.

5A/46 87 Mod. cont.

(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.

5A/47 8-1-1

1. Design shall conform to NCMA - 1979 Sec. 3.3.1, 3.3.2 and Sec.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 50%~~ compressive stresses shall br reduced by 1/3, other stresses shall be reduced bv 1/2.

2. Allowable shear and tension stresses shall conform to **Table 12A-3**, this chapter.

3. Mortar shall conform to NCMA-1979 Sec. 2.2.2.2.

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

5A/48 87/7 (9-0-1)

5. **BEARING STRESS (f_{br})**

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

On one-third 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 reasonably concentric area greater than one-third, but less than the full area shall be interpolated between the values given.

GENERAL

5A/49 87/1 (9-0-1)

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 1/3 for compressive stress, other stresses shall be reduced by 1/2.

(C)1. COMPRESSION IN WALLS AND COLUMNS

A. AXIAL LOADS

5A/50

Stresses 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:

$$F_a = .225\ f'_m\ [1-(h'/40t)^3] \quad \text{walls} \qquad \text{Eq. 12A-1}$$

$$F_a = 0.18\ f'_m [1-(h/30t^3)] \text{ columns}$$

in which:

87/4 (9-0-1)

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

h = (same as p. 149)

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

5A/51

ASSUMED VALUES OF f'_m for use in Eq. 12A-1.

The design ultimate compressive stress of masonry, f'_m, ay 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 Tables 12A-1.

B. BEARING STRESS (f_{br})

On full area, $F_{br} = .25\ f'_m$ Eq. 12A-2

On one-third area or less, $F_{br} = .30\ f'_m$ Eq. 12A-3

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 one-third, but less than the full area shall be interpolated between the values given.

87/6 (9-0-1)

(c)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. (A) and (B).

87/8 (9-0-1)

5A/51 cont.

(c)3. FLEXURAL DESIGN

A. Tensile stresses due to flexural shall not exceed the values given in Section 12A.6.2.(c)3[B] where:

$$f_b = Mc/I \qquad \text{Eq. 12A.4}$$

87/10 (6-3-1)

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

87-10 cont.

Tension Normal to Head Joints

Hollow Units, F_t = 48 psi

Stresses are calculated on net bedded areas

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

(C)4. SHEAR IN FLEXURAL MEMBERS AND SHEAR WALLS

A. SHEAR IN FLEXURAL MEMBERS

$v_m = V/A_e$ Eq. 12A-5

where:

5A/51 cont.

v_m = design shear stress with no shear reinforcement. The allowable shear stresses, V_m, may be equal to

87/12 (9-0-1)

$1.0\sqrt{f'_m}$ 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 reinforcement

5A/52 87/15 (no vote)

$a/L < 1$, $V_m = \frac{1}{3}\left[4 - \frac{a}{L}\right] f'_m$, 50 max. Eq. 12A-6

$A/L \geq 1$, $V_m = 1.0\sqrt{f'_m}$, 35 max. Eq. 12A-7

141 7 of 7

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.

5A/52 cont. 87/15 cont. 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

87/13 (9-0-1) due to horizontal forces. Any resultant tensile stresses shall be resisted by reinforcing in accordance with the requirements of

5A/53 Section 12A.6.3.

(c) 5. <u>CORBELS</u>

The slope of corbelling (angle measured from the horizontal to the face of the corbelled surface) shall not be less than 60°.

87/14 (9-0-1) The maximum horizontal projection of corbelling from the plane of the wall shall not exceed one-half the wythe thickness for cavity walls or one-half the wall thickness for other walls.

5A/54 12A.6.3 DESIGN PROCEDURE FOR REINFORCED MASONRY

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

- A section that is plane before bending remains plane after bending.
- Moduli of elasticity of the masonry and of the reinforcement remain constant.

12A.6.3 Cont.

- 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 homogenous 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:

88 (10-0-0) 1. Running Bond. Six times ~~that~~ the wall thickness. ~~for construction using running bond.~~

89 (10-0-0) 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 stresses shall be proportioned, except as modified by Chapter 12, so that the following formula is satisfied:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad (12A\text{-}7)$$

5A/54 cont.

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 by this Chapter if member were carrying bending load only.

90 (7-1-2) 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:

$$v = \frac{V}{bjd} \qquad (12A\text{-}8)$$

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.

12A.6.3(C) Cont.

91 (10-0-0) j = Ratio of distance between centroid of compression and centriod of tension to depth, d. j = ~~0.8~~ may be assumed as 0.85 or j may be determined by a strain compatibility analysis.

92 (10-0-0) In vertical joints cf stacked bond construction, ~~the masonry~~ 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. ~~Bond beam units shall be used for hollow unit construction.~~

93 (10-0-0) Where the values of the shearing unit stress computed by Formula 12A-8 exceeds the shearing unit stress, v_m, ~~masonry web~~ shear reinforcement shall be provided to carry the entire stress. ~~Web reinforcement shall cross stacked bond joints.~~

94 (10-0-0) 1. ~~Web~~ shear Reinforcement. ~~Web~~ shear reinforcement shall consist of:

a. ~~Stirrups or web~~ Shear reinforcement bars perpendicular to longitudinal steel, or

b. ~~Stirrups or web~~ Shear reinforcement bars archored around or beyond the longitudinal steel and making an angle of 30 degrees or more thereto, or

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

5A/54 cont. d. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of masonry by the reinforcement.

95 (9-0-1) ~~Stirrups or other~~ Bars to be considered effective as ~~web~~ shear reinforcement shall be anchored at both ends.

96 (10-0-0) 2. ~~Stirrups.~~ Required Area. The area of steel, A_v, required ~~in stirrups placed~~ perpendicular to the longitudinal reinforcement shall be computed by the following formula:

$$A_v = \frac{Vs}{f_v jd} \qquad (12A\text{-}9)$$

where V is the total shear, in pounds.

97 (10-0-0) s is the spacing of ~~stirrups or bent~~ bars in a direction parallel to that of the main reinforcement, inches.

98 (10-0-0) f_v is the allowable unit stress in the ~~web~~ shear reinforcement, psi.

99 (10-0-0) Inclined ~~stirrups~~ bars shall be proportioned in accordance with the provisions of paragraph 3 of this Subsection.

100 (10-0-0) 3. Bent Bars. Only the center 3/4 of the inclined portion of any longitudinal bar that is bent up for ~~web~~ shear reinforcement shall be considered effective for that purpose, and such bars shall be bent around a pin having a diameter not less than six times the bar size.

12A.6.3(C) Cont.

101 (10-0-0) When the ~~web~~ shear reinforcement consists of a single bent bar or of a single group of parallel bars all bent at the same distance form the support, the required area, A_v, of such bars shall be computed by the following formula:

$$A_v = \frac{V}{f_v \sin a} \qquad (12A\text{-}9a)$$

where a is 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:

$$A_v = \frac{Vs}{f_v jd\ (\sin a + \cos a)} \qquad (12A\text{-}9b)$$

102 (10-0-0) 4. Spacing of ~~Web~~ shear Reinforcement. Where ~~web~~ 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 ~~web~~ 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.

5A/54 cont.

103 (9-0-1) 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 ~~it across the web~~ and making it continuous with the reinforcement on the opposite face of the member, or anchoring it there.

104 (10-0-0) 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 ~~assist~~ 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 no longer required 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

105 (10-0-0) b. ~~Stirrup area~~ 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 ~~stirrups~~ 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-5 for reinforcement taking no shear. The resulting spacing s shall not exceed $d/8\beta_b$ where β_d is the ratio of the area of bars cut off to the total area of bars at the section, or

12A.6.3(D) Cont.

c. The continuing bars provide double the area required 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.

5A/55 106 (6-4-0) 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 ~~yield strength~~ 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:

$$\frac{M_c}{V} + l_a \qquad (12A\text{-}10)$$

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.

5A/56 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.

107 (8-0-2) 5. Development Lengths. The basic development length, l_d, for deformed reinforcement shall be at least $\cancel{.05\, d_b^2 f_y/\sqrt{f_g}}$ (but not less than 24 d_b for reinforcement of 40,000 psi yield strength nor $36d_b$ for reinforcement over 40,000 psi yield strength, nor less than 12 inches for reinforcing bars and 6 inches for masonry joint reinforcement where: $0.0015 d_b f_s$

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

$\cancel{f_y}$ f_s = the specified bar ~~yield strength~~ calculated stress, psi.

12A.6.3(D) Cont.

107 cont.

~~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:

1. For deformed main compression reinforcement in columns that are not part of the seismic system, these values may be reduced to $18d_b$ for bars of 40,000 psi yield strength and and $27d_b$ 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

5A/56 cont.

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.

12A.6.3(D) Cont.

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.

107A (10-0-0) Lengths of laps, in inches, for deformed reinforcement shall be at least $\cancel{0.08\, d_b^2\, f_y/\sqrt{f_g}}$ $0.002 d_b F_s$ but not less than $40d_b$ 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 $\cancel{f_g}$ F_s shall be as defined in Sec. 12A.6.3(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 $30d_b$ for bars of 40,000 psi yield strength and $45d_b$ for bars over 40,000 psi yield strength.

5A/56 cont. 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.

108 (10-0-0) 8. Anchorage of ~~Web~~ Shear Reinforcement. ~~web~~ Shear reinforcement shall be placed as close to the compression and tension surfaces of the member as cover requirements, practicability, and the proximity of 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 anchored ends, each bend in the continuous portion of a transverse simple- or multiple U-stirrup shall enclose a longitudinal bar, not less in diameter than the stirrup bars.

109 (10-0-0) Longitudinal bars bent to act as ~~web~~ shear reinforcement shall, in a region of tension, be continuous 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.

5A/56 cont. 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.

5A/57 110 (8-0-2) (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.~~

5A/58 111 (8-2-0) 1. Stresses. The axial stress in reinforced masonry bearing walls shall not exceed the value determined by the following formula:

$$f_m = \overset{0.225}{\cancel{0.20}}\ f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right] \qquad \text{(12A-11)}$$

5A/59 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.

112 (9-0-1) h = Effective height or width. 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.

~~2. Reinforcement. Reinforcement of walls and wall elements shall be provided for all loadings and other requirements of these Regulations. Except for the 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.~~

5A/60 113 (9-1-0) 2. Reinforcement. Reinforcement of walls and wall elements shall be provided for all loadings as required by design. Where reinforcement ratios are less than: A. 0.0007 in any direction, B. 0.002 or the sum of the ratios in each direction then; permissible stresses for unreinforced masonry must be used. If reinforcement ratios are equal to or greater than these ratios, the stresses of table 12A-5 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.~~

5A/61 114 (8-1-1) When using the stresses in Table 12.5A, 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 horizonal 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.~~

12A.6.3(E) Cont.

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 be not 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.

5A/62 115 (6-3-1)

The axial load on columns shall not exceed:

$$P = A_g \left(\cancel{0.18}\ 0.20\, f'_m + 0.65\, p_g \cancel{f_s}\, F_s\right) \left[\left(1 - \left(\frac{h}{40t}\right)^3\right]\right. \qquad (12A\text{-}12)$$

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

116 (10-0-0)

p_g $\cancel{\frac{P}{g}}$ = 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.

5A/63

117 (9-0-1)

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

118 (10-0-0)

1. Vertical Reinforcement. The ratio p_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. ~~4~~ 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 rizontal bed joints where permitted by Sec. 12A.3.5(C).

12A.6.3(F) Cont.

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 in size for No. 8 or larger longitudinal bars except that when No. 11 bars are allowed under the exceptions 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.

5A/63 119 (10-0-0) ~~See Chapter 12 for additional requirements, where applicable.~~

120 (10-0-0) 3. ~~Grouting. All columns shall be grouted solid.~~ All column or longitudinal reinforcing shall be solidly embedded in grout.

12A.6.4 MASONRY SHEAR WALLS

121 (10-0-0) The design of masonry shear walls and wall elements for in-plane shears shall conform to this Section, ~~Chapter 12~~, and all applicable provisions of these Regulations. See Chapter 12 for stacked bond construction limitations based on construction categories.

122 (9-0-1) (E) ~~(A)~~ 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.

5A/64 123 (9-1-0) Reinforcement required to resist wall shear shall be terminated with a standard hook ~~which encloses the boundary reinforcing~~ at the end of the wall sections. The hook may be turned up, down, or horizontal and shall be embedded in mortar or grout. Wall reinforcement terminating in boundry columns or beams shall be fully anchored into the boundary elements.

(A) ~~1.~~ INTERSECTING WALLS AND MASONRY COLUMNS. ~~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.

5A/65 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 wall for unreinforced masonry nor eight times the nominal thickness of the intersected wall for reinforced masonry.

Limits on effective 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.

124 (10-0-0) (B) VERTICAL TENSION AND COMPRESSION STRESSES. Except as provided for masonry designed under the alternate design procedure of Sec. 12A.6.2(A) ~~12A.6.2 as modified by Chapter 12,~~ vertical stresses in shear walls shall be determined from the combined effects of vertical loads ~~and from the overturning effects of lateral loads. Minimum vertical loads shall be considered. Formula 3-2a shall be used for unreinforced masonry design.~~

12A.6.4(B) Cont.

Anchorage

125 (8-2-0) Allowable tension stresses for unreinforced masonry shall not be exceeded. ~~Reinforcement anchored~~ to the foundation shall be provided to resist calculated tension in unreinforced walls.

5A/65 cont. " (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 ~~piers. For unreinforced masonry, allowable shear and tensile stresses shall not be exceeded. Tensile~~
126 (9-0-1) ~~reinforcing and shear reinforcing, if required, shall be provided for reinforced masonry. In reinforced masonry, when the horizontal span of the element is less than twice the total height of the element, shear reinforcing shall be in the form of diagonal bars extending from corner to corner with complete anchorage to the pier elements or shall be web reinforcing conforming to Sec. 12A.6.3(C).~~

5A/66 127 (9-1-0) (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.

minimum

Shear resistance of masonry shall be based on minimum net areas parallel to the shear. Both vertical and horizontal shear shall be considered. ~~including the net bedded area, the net cross-sectional area of hollow units, and the net vertical shear area. Where only partial mortar coverage is provided, such as in hollow unit construction where only the face shells in the bed joints and partial head joint coverage is usually specified, only the actual specified mortar coverage shall be considered effective. However~~, Continuous vertical and horizontal grout elements may be considered as part of the net areas.

For reinforced masonry, the shear stress shall be computed by Formula
128 (10-0-0) 12A-8. 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 or as required by Sec. 12.7, whichever is less.

~~Reinforcement required to resist wall shear shall be terminated with a standard hook which encloses the boundary reinforcing of wall sections. The hook may be turned up, down, or horizontal 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

5A/67 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.

129 (10-0-0) The panels shall be capable of spanning between supports to resist horizontal forces. Wind loads shall be based on gross projected area of the ~~block.~~ panel

130 (10-0-0) ~~The maximum size of panels shall be 144 square feet with the maximum dimension in either direction of 15 feet. Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel. Supports at the top and ends of the panel shall be by means of confinement of the masonry by at least 1/2 inch into and between the flanges of a steel channel. The space between the end of the panel and the web of the channel shall be at least 1/2 inch and shall be void of mortar. The use of equivalent configuration in other steel sections or in masonry or concrete is acceptable.~~

131 (10-0-0) ~~Horizontal and vertical joints shall be not less than 1/4 inch thick. All joints shall be completely filled with mortar and shall be shoved joints.~~

(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).

12A.6.5 Cont.

(B) REINFORCED PANELS. All panels in Categories B, C, or D construction shall be reinforced per Sec. 12.4.1(D).

132 (9-0-1)

Sec. 12A.7 ~~SPECIFIC INSPECTIONS, SPECIAL~~ INSPECTIONS, AND TESTS

133 (9-0-1)

~~Specific and Special~~ 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, ~~Specific~~ Inspections, Tests, ~~Special Inspection~~, or some items of Special Inspection. ~~The Special Inspections and Tests of Sec. 12A.7.2, where applicable, shall be provided for all parts of masonry construction. The Special Inspection requirements of Sec. 1.6.2 are in addition to Sec. 12A.7.2 and apply only to the designated seismic system.~~

134 (9-0-1)

~~Specific and Special~~ Inspection shall be done to an extent that the Inspector(s) or testing agency can certify to the requirements of Sec. 1.6.4. ~~In general, for large jobs or for moderate size jobs, this will require continuous observation during the masonry work. However, some inspections may be done on a periodic basis provided they satisfy the requirements of this Chapter and provided this periodic scheduled inspection is performed as outlined in the project design documents or the approved Quality Assurance Plan.~~

5A/67

135 (9-0-1)

12A.7.1 ~~SPECIFIC~~ FREQUENCY OF INSPECTIONS AND TESTS

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

- 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.

~~The Specific~~ 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 certifications 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 observation of the laying of masonry units with special attention to joints including preparations prior to buttering, portions to be filled, shoving, etc.

12A.7.2 Cont.

5A/67 cont.

- For observation of the bonding of units in the walls between wythes and at corners and intersections.
- For the proper placement 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.

5A/68 136 (10-0-0)

(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 or type of masonry construction. ~~except for the total number for a building.~~

5A/69 137 (9-0-1)

- For masonry units. When shipments of masonry units are not identified and accompanied by certification ~~acceptable to the Regulatory Agency,~~ 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.

5A/70 138 (8-2-0)

- ~~For unreinforced or reinforced~~ Seismic Performance Category D, grouted masonry, one series of core tests for shear bond shall be made for each 5000 square feet of wall or equivalent. Move to Ch. 12

12A.7.2 Cont.

- 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 Sections 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:

5A/71

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

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

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.

12A.8.1 Cont.

Prisms shall be built of the same materials, under the same conditions, and, insofar as possible, with tne same bonding arrangements as for the structure includinq 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 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.

5A/71 cont. 447 (A) STORAGE OF TEST PRISMS. For storage of test prisms follow Method B of ASTM ~~E477~~ 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.

139 (8-0-2) Test prisms made in the field shall be stored undisturbed and protected from freezing and excessive drying for 48 ~~to 96~~ hours in the field under the same conditions, insofar as possible, and adjacent to the work they are to represent. ~~They may be covered with wood or damp burlap, but such covering shall not shade the sides from the sun.~~ After field storage, they shall be transported to the laboratory for continued curing as specified for laboratory constructed prisms. ~~Field curing may continue as specified for the initial seven days.~~

Test prisms and cores cut from the work shall not be taken before the work is seven days old. Prisms cut from the 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.

140 (10-0-0) ~~Prisms shall be capped and tested in compression. The standard age of test specimens shall be 28 days, but 7-day tests may be used provided the relation between the 7-day and 28-day strengths of the masonry is established by adequate test data for the materials use~~d.

12A.8.1 Cont.

(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.

140A-New (8-2-0)

12A.8.2 ~~TESTS FOR GROUT AND MORTAR~~ 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 condition.

5A/71 cont.

141-Mod (7-3-0)

~~(B) MORTAR SAMPLES FOR COMPRESSION TESTS. Spread mortar on the masonry units 1/2-inch to 5/8-inch thick. Place a masonry unit on top of the mortar and allow to stand for two minutes. Immediately remove mortar and place in a 2-inch by 4-inch cylinder in two layers, compressing the mortar into a cylinder using a flat-end stick or fingers. Lightly tap mold on opposite sides, level off, and immediately cover molds and keep them damp until taken to the laboratory. After 48-hours set, have the laboratory remove molds and place them in the fog room until tested in the damp condition.~~

(B) FIELD MORTAR SAMPLES FOR COMPRESSION TESTS. Spread a ½ 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 containes 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 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.

141A-New (8-2-0)

(D) COMPRESSION TESTS. Excluding curing, storage, and test age requirements, compression testing procedures 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.

5A/72

141B-New (6-4-0)

~~(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.

5A/73

(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 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.

12A.8.3 Cont.

142 (7-1-2) (C) PROCEDURES. The wall shall be at least 14-days old before cores are taken. Cores shall be tested at ~~approximately~~ 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.

5A/73 cont.

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

143 (7-1-2) ~~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. Move to Ch. 12

144
(10-0-0)

TABLE 12A-1A

COMPRESSIVE STRENGTH OF MORTAR CLASSIFIED IN ACCORDANCE WITH PROPERTY SPECIFICATIONS (Pounds per Square Inch)

Mortar Type	Average Compressive Strength at 28 Days
M	2,500
S	1,800
N	1,500

Both tables are covered by ASTM C270

5A/73
cont.

145
(10-0-0)

TABLE 12A-1B

MORTAR PROPORTIONS FOR MORTAR CLASSIFIED IN ACCOROANCE WITH PROPORTIONS CLASSIFICATIONS (Parts by Volume)

Mortar Type	Portland Cement	Masonry Cement	Hydrated Lime or Lime Putty[1]	Aggregate Measured in a Damp, Loose Condition
M	1	1	--	Not less than 2½ and not more than 3 times the sum of the volumes of the cements and lime used.
	1	--	¼	
S	½	1	--	
	1	--	over ¼ to ½	
N	--	1	--	
	1	--	over ½ to 1¼	

[1]When plastic or waterproof cement is used as specified in Sec. 12A.1.14, hydrated lime or putty may be added but not in excess of one-tenth the volume of cement.

146
(void)

5A/74 147 (7-1-2)

TABLE 12A-2

~~MINIMUM THICKNESS OF MASONRY WALLS~~

MAXIMUM HEIGHT TO THICKNESS RATIOS

5A/75 148 (9-1-0)

5A/76 149 (10-0-0) (8-2-0)

TYPE OF MASONRY	Maximum ~~thickness~~ ratio on unsupported height or length to Thickness[1]	NOMINAL MINIMUM THICKNESS (INCHES)[3]: Walls whose only structural function is exterior enclosure, nonstructural walls, and partitions	Other Structural Uses: Thickness for the uppermost 35[6] foot high portion of wall	Other Structural Uses: Thickness increase for each 35 feet or fraction thereof below the uppermost 35[6] foot high portion of wall
STRUCTURAL WALLS:				
Unburned Clay Masonry	10	16	16	--
Stone Masonry	14	16	16	4
Cavity Wall Masonry	20[5]	8	12[7]	4
Hollow Unit Masonry	20	8	12[7]	4
Solid Masonry	20	8	12[7,8]	4
Grouted Masonry	20[2]	6	10[7,8]	4
Reinforced Grouted Masonry	~~25[2,9]~~ 36[2]	6	6	--
Reinforced Hollow Unit Masonry	~~25[2,3]~~ 36[2]	6	6	--
NONSTRUCTURAL AND PARTITIONS:[4]				
Unreinforced	36[5]	2	--	--
Reinforced	48	4	--	--

5A/77

[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 grouteo masonry and 36 for reinforced walls.

150 (8-0-2)

~~[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.

150A -New (7-2-1)

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

$$\cancel{T = 2/3\ (T_0 - w_c)}\quad T = \sqrt{t_1^2 + t_2^2}$$

$T =$
$t_1 =$
$t_2 =$
$h =$

where ~~T_0 = overall thickness of wall, including width of cavity,~~

~~w_c = width of cavity~~

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

151 (8-0-2)

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.

152 (8-0-2)

~~[6]Seventy feet for stone masonry.~~

~~[7]These thicknesses may be reduced to 8 inches for walls that are not over 35 feet in total height in buildings that are not over three stories high.~~

153 (8-0-2)

~~[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.~~

154 (7-1-2)

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

5A/77 cont.

TABLE 12A-3

ALLOWABLE WORKING STRESSES IN UNREINFORCED MASONRY

MATERIAL[6]	MORTAR TYPE: M	S	M OR S				N		
	Compression[1]	Compression[1]	Shear or Tension in Flexure[2,3,8]		Tension in Flexure[3,4,8]		Compression[1]	Shear or Tension in Flexure[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	140	12	6
Grade S	125	115	12	6	24	12	100	12	6
Grouted Masonry, Multiwythe with Solid Units									
>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[5]	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 gross on 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-2 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 certifications 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.

5A/77 cont. 87/2 (9-0-1)

TABLE 12A-4

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]	4300[2,5]	5300[2,5]	6300[2,5]
	12,000 psi gross[1]	3800[2,5]	4600[2,5]	5500[2,5]
	10,000 psi gross[1]	3300[2,5]	4000[2,5]	4600[2,5]
	8,000 psi gross[1]	2700[2,5]	3300[2,5]	3800[2,5]
	6,000 psi gross[1]	2200[5]	2600[5]	3000[2,5]
	4,000 psi gross[1]	1600	1900	2200[5]
	2,000 psi gross[1]	1100	1200	1300
Solid Concrete ~~Units other than Clay~~ and Net Area of Hollow Concrete	6,000 psi gross	1350	~~1900~~ 2400	2400
	4,000 psi gross	1250	~~1650~~ 2000	2000
	2,500 psi gross	1100	~~1350~~ 1550	1550
	1,500 psi gross	875	~~1025~~ 1150	1150
	1000	700	900	900
~~Hollow Concrete~~	~~Gd. N~~	~~--~~	~~1350~~	~~1350~~
~~Hollow Concrete - Grouted Solid~~	~~Gd. N~~	~~--~~	~~1500~~	~~1500~~
~~Hollow Clay~~	~~Gd. LB with 1-¼" Min face Shell~~	~~--~~	~~1350~~	~~1350~~
~~Hollow Clay - Grouted Solid~~	~~Gd. LB with 1-¼" Min face Shell~~	~~--~~	~~1500~~	~~1500~~
~~Hollow Clay Brick~~	~~5,000 psi net[1]~~	~~--~~	~~2500[5]~~	~~2500[5]~~
~~Hollow Clay Brick - Grouted or Reinforced~~	~~Type I~~	~~--~~	~~2000~~	~~2000~~

[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 certifications 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-S

ALLOWABLE WORKING STRESSES (PSI) FOR REINFORCED MASONRY

Amendment	TYPE OF STRESS	REINFORCED GROUTED AND HOLLOW UNIT MASONRY SPECIAL INSPECTION REQUIRED: YES	NO
5A/78 155A-New (6-4-0)	Compression-Axial, Walls	See Section 12A.6.3(E)	~~One-half~~ 2/3 of the values permitted under Section 12A.6.3(E)
155B-New (6-4-0)	Compression-Axial, Columns	See Section 12A.6.3(F)	~~One-half~~ 2/3 of the values permitted under Section 12A.6.3(E)
156-Mod (8-1-1)	Compression-Flexural	0.33 f'_m but not to exceed ~~900~~ 2000	0.166 f'_m but not to exceed ~~450~~ 1000
	Shear:		
157 (void)	Reinforcement taking no shear[3]		
	Flexural[4]	1.1 $\sqrt{f'_m}$ 50 Max.	25
	Shear walls[3,4] M/Vd ≥ 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 ≥ 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
5A/79	Modulus of Rigidity	240 f'_m but not to exceed 1,200,000	200 f'_m but not to exceed 600,000
158-Mod (9-0-1)	Bearing on full Area[5]	0.25 f'_m but not to exceed ~~300~~ 1500	0.125 f'_m but not to exceed ~~450~~ 750
	Bearing on 1/3 or less of area[5]	0.30 f'_m but not to exceed ~~1200~~ 1800	0.15 f'_m but not to exceed ~~600~~ 900

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

159 (10-0-0) [2]~~Web~~ reinforcement shall be provided to carry the entire shear in excess of 20 pounds 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 ¼ 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-6

ALLOWABLE SHEAR ON BOLTS[1,4]

DIAMETER OF BOLTS (Inches)	UNBURNED CLAY UNITS: MINIMUM EMBEDMENT (Inches)	UNBURNED CLAY UNITS: SHEAR (Pounds)[2]	ALL OTHER MASONRY: MINIMUM EMBEDMENT (Inches)	ALL OTHER MASONRY: SHEAR: SOLID MASONRY (Pounds)[2]	ALL OTHER MASONRY: SHEAR: 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 are for construction where Special Inspection is not provided. Where Special Inspection is provided 150 percent of these values are permitted.

5A/80 160 (8-1-1) [3]These values are permitted only with units having a minimum net area compressive strength of 2500 pounds per square inch or more.

5A/81 161 (8-0-2) 4An anchor bolt is a bolt that has a right angle extension of at least 3 diameters. A standard machine bolt is acceptable.

162 - New
(9-0-1)

TABLE 12A-7

ALLOWABLE STRESSES TO BE USED WITH THE
ALTERNATE DESIGN PROCEDURE FOR UNREINFORCED BRICK MASONRY[1]

DESCRIPTION		ALLOWABLE STRESSES, PSI WITHOUT SPECIAL INSPECTION	ALLOWABLE STRESSES, PSI WITH SPECIAL INSPECTION
Compressive, Axial[2]			
Walls	f_m	0.13 f'_{mb}	0.20 f'_{mb}
Columns	f_m	0.10 f'_{mb}	0.16 f'_{mb}
Compressive, Flexural[2]			
Walls	f_m	0.21 f'_{mb}	0.32 f'_{mb}
Columns	f_m	0.17 f'_{mb}	0.26 f'_{mb}
Tensile, Flexural[5,6]			
Normal to bed joints[2]			
M or S mortar	f_t	18	36
N mortar	f_t	14	28
Parallel to bed joints[3]			
M or S mortor	f_t	36	72
N mortar	f_t	28	56
Shear[7]			
M or S mortar	V_m	$0.3\sqrt{f'_{mb}}$ but not exceed 35	$0.3\sqrt{f'_{mb}}$ but not to exceed 75
N mortar	V_m	$0.3\sqrt{f'_{mb}}$ but not to exceed 28	$0.3\sqrt{f'_{mb}}$ but not to exceed 56
Bearing			
On full area	f_m	0.17 f'_{mb}	0.25 f'_{mb}
On one-third area or less[4]	f_m	0.21 f'_{mb}	0.375 f'_{mb}
Modulus of Elasticity	E_m	500 f'_{mb} but not to exceed 1,500,000 psi	600 f'_{mb} but not to exceed 3,000,000 psi
Modulus of Rigidity	E_r	200 f'_{mb} but not to exceed 600,000 psi	240 f'_{mb} but not to exceed 1,200,000 psi

[1]See Section 12A.6.2.
[2]Direction of stress is normal to bed joints; vertically in normal construction.
[3]Direction of stress is parallel to bed joints; horizontally in normal masonry construction. If masonry is laid in stacked bond, tensile stresses in the horizontal direction shall not be permitted in the masonry.
[4]This increase shall be permitted only when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area shall be interpolated between the values given.
[5]For computing the flexural resistance of cavity walls, the lateral load shall be distributed to the wythes according to their respective flexural rigidities.
[6]In the use of these allowable stresses, consideration shall be given to the influence of unusual vibration and impact forces.
[7]See Section 12A.6.3(c).

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/1	• Definition - The new definition for net area is a clarification. • Definition - The definition for grouted masonry clarifies the one proposed for deletion in ballot item 5A/4. A yes vote here should have a yest vote on 5A/4.	Yes - This just clarifies some definitions.
5A/2	• Deletion - The definition is unnecessary. Load bearing is an accepted term; however, all structural walls may not be load bearing (e.g., shear walls).	No - Editorial revision must be assured. Preferable to leave as is. Term "load bearing" is used in other places.
5A/3	• Insert - These new definitions are consistent with the text as revised. Specific allowable stresses are defined in the text. The committee prepared the text, especially ballot item 5A/60 prior to preparing 5/3 which is similar for seismic. Item 5A/60 needs to be checked for consistency with 5/3 after ballots are comted. A yes vote here should have a yes vote on 5A/6.	Yes, with explanation A yes vote carries the implication that Ch. 12A be editorially revised so that the definition for reinforced masonry is consistent with the definitions given in 12.2.1(B). These definitions state when reinforced and unreforced allowable stresses can be used.
5A/4	• Deletion - The definition was clarified in item 5A/1 which replaces 5A/4. A yes vote here should also have a yes vote on 5A/1.	Yes - This definition has now been covered under ballot item 5A/1.
5A/5	• Deletion - The definition of non-bearing does not add to the currently accepted definition. The term non-load bearing is contradictory since they may be structural elements.	No - Same reason for disapproval as 5A/2.
5A/6	• Deletion - This definition should be deleted since it is not consistent with the recommendation of item 5 A/3. A yes vote on 5A/3 should have a yes vote here.	Yes - No objection as long as the document clearly delineates the difference between "reinforced" stresses and other conditions of less reinforcement. Important that it be understood that the term "partially reinforced" described masonry with less than prescribed minimum reinforcing ratios and hence did not qualify it for using "reinforced" stresses. See also 5A/3.
5A/7	• Deletion - same as 5A/6	Yes - see 5A/3 and 5A/7
5A/8	• Partial deletion - Minor editorial revision	Yes - Minor editorial revision

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/9	• Insert - Editorial modification and specification update and addition.	Yes - Minor editorial expansion.
5A/10	• New insert - The inserts make the revised text and symbols consistent.	Yes - Changes were minor and have no real objection. Item 16 - would be difficult to visualize a "length" of column that is governing rather than height. Item 17 - term p should actually be p_g.
5A/11	• Deletion and editorial modification - The deleted item was considered seismic and therefore was moved to Ch. 12	Yes - This requirement has been transferred to SPC-'C'.
5A/12	• Modification - The change from 3.5 to 3.0 was felt more appropriate for the material as used.	Abstain - Didn't feel the amount of construction involved had any real impact.
5A/13	• Deletion - Shrinkage control is considered a serviceability, not a safety or seismic item. Shrinkage properties of concrete masonry units are already adequately covered in the referenced ASTM specifications.	Abstain - Abstained with the idea that it would be included in SPC-'B', however, we made an ommission error in not picking this up until SPC-'C'.
5A/14	• Modification - The modification is to use a more appropriate ASTM reference with an additional statement so that only portland cement can be used. • Deletion - The "exception" is not needed with the above modification.	No - ASTM C270 limits cements for mortar, not grout. It does not allow type V cement, which can be used for grout. It does allow masonry cement, which is not suitable for grout. If this change is allowed, Sec. 12A.1.17(B) should be revised to permit only portland cement or blended hydraulic cement. Note that the UBC and BIA codes allow only portland cement for grout.
5A/15	• Partial deletions and inserts - All are minor editorial changes.	Yes - No objection to minor changes.
5A/16	• Partial deletion - Grout consistency should match job requirements. The designer may be misled by slump limits. Lower slump limits are not better than larger ones for grout.	No - Felt it is important construction consideration to have limits (upper and lower) on grout consistency.
5A/17	• Deletion - A "drum type batch mixer" is only one type of suitable mechanical mixer. ASTM C94 does not refer to mix uniformity.	No - Performance criteria will allow the inspector to order obviously defective equipment off the job.
5A/18	• Deletions and inserts - The insertions are briefer improvements over the deleted material. Other deleted material is not needed.	Yes - Felt dampening of units for certain weather conditions was desirable but back-off on this in "spirit of compromise"

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/19	• Partial deletion and insert - The deleted material is a laundry list, the insert is more appropriate and concise.	No - Floor slabs or spandrel beams within walls should not be treated different from foundations. Highly stressed masonry in columns and pilasters should be continuous for the vertical load.
5A/20	• Deletion - The material is adequately covered with the insert in 5A/21. The amplitudes given cannot be enforced. The item was added in SPC-B.	Yes - We now have no objection since this was picked up in SPC-'B'.
5A/21	• Insert - This item must be inserted with the deletion in the previous ballot.	Yes - Desirable construction requirement.
5A/22	• Deletions and inserts - These are editorial revisions	Yes - Minor editorial revision.
5A/23	• Partial deletion - A template is only one way to achieve the performance requirement to accurately set bolts. The template is specified in Ch.12.	No - "Templates" provides a standard for "approved equivalent means." "Dunking" and "stabbing" anchor bolts is prevalent practice (abuse); it should be controlled.
5A/24	• Modification - The 2.5-inch requirement may require special cutting since units do not come in multiples of 2.5 inches. The 4-inch dimension is compatible to standard units.	No - Five-eighths and smaller bolts require only 4-in. embedment. Ties 4 in. down will not be effective.
5A/25	• Partial deletions and inserts - These are all editorial.	Yes - Minor editorial revision.
5A/26	• Partial deletion and insert - This change allows the use of multi-wythe grouted walls in common and satisfactory usage in many parts of the U.S., including California. Solid units will use solid unit stresses and hollow units will use hollow unit stresses.	No - A no vote would revert this to the original definition. Proposed wording could permit defining almost any type of masonry construction as "grouted masonry" and to include any types of units such as hollow, tile, etc. ATC wording defines "grouted masonry" consistent with provision of UBC, SBCC, and common usage. "Allowable stresses" are for the ATC definition which should be retained. Allowables for other possibilities have not been developed.
5A/27	• Deletion - This was felt to be needed only for seismic so it was moved to Ch. 12.	Yes - No further objection since this has now been incorporated in SPC.'B'.
5A/28	• Deletions and insertions - All are editorial.	Yes - Unobjectionable editorial changes.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/29	• Partial deletion - The deletion requires cleanouts only as necessary for a job to insure cleanliness. It is performance rather than specification oriented.	No - Leaving out every other unit follows approved 1976 UBC code change wording by the masonry industry. Fin and foreign matter removal procedures are suggestions, conforming to the ACI 531 Commentary, as to what is required. This could be removed; however, the word "other" allows alternatives. Successful high-lift work requires the special procedures of this paragraph and cleaning procedures should have a basic standard.
5A/30	• Editorial changes	Yes - Unobjectionable changes.
5A/31	• Partial deletion - The material remaining after the deletion is all that is necessary to cover embedded reinforcement, both horizontal and vertical. All reinforcement, not just vertical, must be suitably anchored.	No - Unless the principal bar reinforcing is embedded in grout, the form of construction will be other than that commonly known as "reinforced grouted masonry--multiple wythe" and that tested as such. This is the only form of construction shown in the BI of California handbook and the ACI 531 Commentary as "reinforced grouted masonry--multiple wythe."
5A/32	• Insert - Additional aggregate requirements are included.	Yes - No objection.
5A/33	• Deletion - The paragraph is unnecessary considering the redefinition of reinforced masonry.	No - This paragraph is needed so addition of steel in just any fashion to hollow unit construction will not qualify the construction as being reinforced. The construction described here and in Sec. 12A.3.6(A) following conforms to that commonly known as "hollow unit work" and that tested as such. The paragraph should be retained and will be included in the editorial redefinition of reinforced masonry.
5A/34	• Title change - The revised title is a more appropriate description of the content of the paragraph.	No - Title: There may be a real problem with the title. Sec. 12A.3.6, following the UBC, recognized only ungrouted unreinforced work. Allowable stresses of Table 12A-3 apparently are only set up for this format. If 12A.3.6 is to allow unreinforced grouted hollow unit construction, then more revisions may be required in the allowable stresses and other areas.
5A/35	• Editorial changes.	Yes - Minor editorial and reinserting previously removed items.
5A/36	• Deletion - the paragraph suggests construction in which it is difficult to clean out debris. It restricts grout lifts which have been successfully used on numerous occasions.	No - Need some limits. ACI Commentary Sec. 6.4 places the limit at 12 ft. The ATC provision is more liberal and should be retained.
5A/37	• Partial deletion - refer to comment on 5A/29	No - Same comment as 5A/29

Joint Committee Ballot No.		Type of Change and Committee Comment	ATC Recommendation and Comments
5A/38	•	Deletions - The deletions are necessary for consistency with other changes in definitions of reinforced masonry	Yes - Changes made for consistency with other approved changes.
5A/39	•	Modification - Refer to 5A/12	Abstain - Same comment as 5A/12
5A/40	•	Partial deletion and editorial.	Yes - Item 76 is just reinsertion of an item. Other change is minor editorial.
5A/41	•	Partial deletions, inserts - The deleted material is covered by the reference to table 12A-2.	Yes - (with explanation) A 'yes' vote on 78 was made with the understanding that Table 12A-2 would be referenced. A later change then substantially changed Table 12A-2. Therefore, in light of this development, reference should be made to the items of Table 12A-2.
5A/42	•	Partial deletions and inserts - Editorial changes	Yes - Essentially editorial changes

The current ATC 3-06 sec. 12A.6.2 covers only solid clay. The new 12A.6.2 (items 5A/43 through 5A/53) covers solid clay, concrete masonry and hollow clay. A 'yes' ballot means take the text as shown. 'No' means taken with the ATC recommendation.

5A/43	•	Alternate design procedures - New introduction to bring in new materials.	Yes - This is only down to Section 12A.6.2(A) 5a.
5A/44	•	Insert - Part of new material. The material shown will make uninspected masonry consistent with provisions in existing design standards.	No - This 'no' vote will change all permissible stresses for uninsp'd brick masonry to 50% of the permissible stresses for insp'd brick masonry. And this is to be consistent with our other recommendations for other materials. We believe it is very important because of the potential quality control with uninspected masonry.
5A/45	•	Introduces new material	Yes - Continuation of item 5A/43 that is from 12.A.6.2(A)6.
5A/46	•	Introduces lead-in to new material.	Yes - This pertains to 1st paragraph only in which there was no change.
5A/47	•	Modification of new insert - The stress reductions shown are consistent with ACI 531 for design of masonry.	No - This pertains to 12.A.6.2(B)1 only. Same comment as 5A/44.
5A/48	•	New insert - Provide guidance not in ATC 3-06.	Yes - This pertains to 12.A.6(B)2,3,4,5
5A/49	•	New insert - Provides guidance not in ATC 3-06. Wording provides for allowable stresses consistent with other materials.	No - This pertains to 1st paragraph "GENERAL". This is to be consistent with other permissible stresses for other material. See 5A/44.
5A/50	•	New insert - Provides guidance not in ATC 3-06. The coefficient is consistent with that recommended for other materials.	No - Our 'no' vote infers that the 0.225 constant of Eq. 12A-1 should be 0.20. We missed this on the original vote.
5A/51	•	New insert - Provides guidance not in ATC 3-06.	Yes - Pertains to everything from Eq. 12A-1 to 12A.6.2(C)4A. Had no objection to changes.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/52	• New Insert - Provide guidance not in ATC 3-06. Due to lack of time, this is the only item that was not formally balloted by the entire committee. The proponent states that this is the best currently available data. The committee agrees with ATC on the need for an examination of fundamental data forming the basis for allowable shear stresses. Such an examination is unlikely to be completed before trial designs. Stresses used in this document are those which are currently accepted.	No - A 'no' vote infers a re-examination of the allowable shear stresses for all three (materials) of unreinforced construction by Committee 5. This item was never resolved during Com. 5 deliberations.
5A/53	• Minor editorial changes.	Yes - Minor editorial changes
5A/54	• Minor editorial changes	Yes - Primarily minor editorial changes
5A/55	• Editorial revision - the paragraph shown as item 106 was revised to base anchorage on a working stress basis for compatibility with the rest of Ch. 12A. The strength-based development length was moved to the strength-based Ch. 12. In making this change, a minimum anchorage length was inadvertently omitted. The committee agrees that the need for such a minimum should be examined prior to trial designs. Anchorage based on yield is a ductility requirement for seismic.	Yes - It was felt that a min. anchorage length should be stipulated. Therefore use anchorage to develop calculated stress or a minimum specified anchorage length, whichever is greater.
5A/56	• Minor change to convert to working stress based design.	Yes - Had no objections to proposed changes.
5A/57	• Deletion - The height/thickness ratios are more appropriately controlled by the interaction equation 12A-7.	No - Should be retained since it removes the height/thickness ratios which we believe are important.
5A/58	• Partial additions and inserts - The major item is the coefficient 0.225 in eq. 12A-11. The change to 0.225 is consistent with existing standards such as ACI 531 and NCMA. There has been considerable laboratory and field experience to justify these standards. It should be noted that the committee did agree to retain the 0.20 for the seismic Ch. 12 but not this nonseismic chapter.	No - If this change in coefficient is passed, it leads to greater allowables in reinforced masonry walls than in reinforced concrete walls. The concrete walls have a greater amount of reinforcement (0.004 as compared to 0.002) for masonry. At a slenderness ratio of 25, ACI Eqtn 14-1 for walls reduces to $0.134\ f'_c$ using the alternate (working stress) method, and to $0.151\ f'_c$ using a u factor, averaged for dead and live loads at 1.55. At the same slenderness ratio the proposed ballot item would allow a masonry stress of $0.170\ f'_m$. The factor of 0.20 is used in UBC but 0.225 is used in NCMA and ACI.

Joint Committee Ballot No.		Type of Change and Committee Comment	ATC Recommendation and Comments
5A/59	•	Editorial for consistency	Abstain - Actually redundant. This was covered under 5A/10.
5A/60	•	Insert - Based on new definition of reinforced masonry. It is intended to provide reinforcement limits as to which tables of stresses can be used. The committee noted that this item was prepared about a month before the more comprehensive item 5/3 in Ch. 12. Following balloting it should be examined for consistency.	No - The 'No' vote infers that the original wording of ATC-3-06 should be retained so that spacing and continuity of horizontal reinforcement is covered. This is another item that needs to be editorially revised in light of the new definitions for reinforced masonry.
5A/61	•	Partial deletion - The wording change was a clarification. The existing text is confusing on what to do with steel in multiple unit thickness walls.	No - The changes would remove commonly accepted practices.
5A/62	•	Modification - Refer to comment on 5A/58. The reason is the same. The lower coefficient was retained in Ch. 12 for seismic.	No - The 'No' votes infers that orig. ATC-03-06 coefficient should be retained. For columns in Item 115, if a comparison is made of short 12 in. x 12 in., columns with no eccentricity, $f'_m = f'_c$ = 6,000 psi and 4 - #10 Grade 60 re-bars, the masonry column is allowed 87% of the load of a concrete column calculated with the alternate working stress method. This spread should be greater and, in addition, traditionally, masonry codes have allowed about 30% of wall loads for the design of columns.
5A/63	•	Editorial changes	Yes - Minor changes
5A/64	•	Partial Deletion - This was believed to be a seismic requirement if a requirement at all. Most did not readily see how the hook could be detailed in the prescribed manner.	No - This requirement for enclosing the boundary reinforcement is a desirable confinement of the vert. boundary bars and should be retained.
5A/65	•	Partial deletion and insert - clarifies text.	Yes - Agree with change.
5A/66	•	Deletion - The original wording is not clear. The new wording is an improvement but may also not be as clear as it should be. It is agreed that the wording should be examined again to avoid misinterpretation.	No - For reinforced walls shear should be based on an effective depth or jd. In Section 11.10.4 of ACI 318 the effective depth of the reinforced wall is specified as 0.8 ℓw or an effective depth determined by a strain compatibility analysis. If this item passes it effectively increases the allowable shear stresses by approximately 20% for reinforced masonry. The original wording needs to be revised to convey the intent.
5A/67	•	Partial deletion and Inserts - Editorial and clarification.	Yes - No significant objections to the proposed changes

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comments
5A/68	• Partial deletions - The deleted material is project specifications for quality control and not specifications to assure safety. It is not for design.	No - As the content of the first bullet has been amended, field prism tests are only required when f'm is to be established by prism test and special inspection stresses are used. The significant change from what was originally required is that no quality control is required when f'm is assumed from Table 12A- and special inspection stresses are used. The assumed values of f'm can be as high as 6,300 psi and the area of grout can be large in proportion to the total cross sectional area for some elements, especially columns, and there is no check on the pris or grout strength.
5A/69	• Editorial	Yes - Change was minor
5A/70	• Move to seismic Ch. 12	Yes - Had no objection when it was agreed to move this to Chap. 12
5A/71	• Editorial and reference to ASTM. Committee item 141 is on ATC proposal for revision.	Yes - Minor changes.
5A/72	• Deletion - Covered by ASTM specifications.	No - Deletion of this item removes any requirement for minimum strengths mortar and grout.
5A/73	• Editorial - ATC agrees that the deleted table 12-1 is covered by ASTM specs and need not be repeated.	Yes - No objections to changes.
5A/74	• Deletion - The thickness of a wall will be controlled by the limiting h/t restrictions and masonry units available for use. Therefore, arbitrary minimum thicknesses are not needed. The changes made in the text require the changes in the table.	No - (with explanation) A 'No' vote implies the retention of the Min. Thickness Requirements except that the last two columns will be removed.
5A/75	• Editorial	Yes - This should read "Unsupported Height". . . .
5A/76	• Change - the modifications made the h/t requirements consistent with existing standards.	No - The height to width ratio has been arbitrarily increased from 25 to 36 for reinforced walls with no backup test data. Both UBC and ANSI A41.2 use 25 and NCMA uses 30; ACI 531 uses 36.
5A/77	• Inserts and deletions - The change (150A) is one of several possibilities suggested by ATC. Others are for consistency.	Yes - No serious objections to proposed changes.
5A/78	• Coefficient change - Refer to 5A/44	No - Refer to 5A/44 for comments and as follows: The ATC document uses the most commonly used reductions on allowable stresses for uninspected masonry construction. If this ballot item passes and ballot items 111 and 115 pass, then allowable compressive stresses for work without special inspection will be 50% greater than stresses currently used.

Joint Committee Ballot No.		Type of Change and Committee Comment	ATC Recommendation and Comments
5A/79	•	Minor change	Yes - minor changes
5A/80	•	Insert - Net area strength is used in design and the insert makes the design procedure consistent.	No - Both ASTM C90-75 and C652-75 use gross area strengths for hollow units.
5A/81	•	Minor changes	Yes - Minor change.

BACKGROUND

5/1 12-1/M-1 (10-0-0)

The masonry design and construction procedures given in this Chapter and Chapter 12A are essential to ~~providing the~~ provide seismic performance levels ~~implicit in the selection of the factors used in determining the seismic forces in these provisions. The requirements embodied in Chapters 12 and 12A have been demonstrated to be necessary by recent earthquakes and represent the latest developments in masonry construction to provide adequate seismic performance.~~ 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

M-2 (10-0-0)

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 allowables are further modified by this chapter. The value of φ shall be as follows:

5/2

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

12-1AN (10-0-0)

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. If f'_m is to be established by test, a minimum of three prism test series (as defined in 12A.8.1(B)) shall be made during the progress of the work.

12.2.1 SPECIAL DESIGN PROCEDURES FOR ~~UNREINFORCED~~ MASONRY SUBJECTED TO SEISMIC FORCES.

5/3 12-2N (10-0-0)

~~Unreinforced~~ Masonry shall be designed in accordance with this Section.

(A) ~~GENERAL~~ UNREINFORCED DESIGN PROCEDURE. 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-3.

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.

(3) REINFORCED MASONRY DESIGN. Reinforced masonry shall be designed and constructed in accordance with one of the following procedures and the provisions of other Sections of this Chapter.

1. Masonry designed and reinforced as required.

5/3 cont. 2. Masonry designed and reinforced as required and containing nominal prescribed reinforcing. Construction shall be grouted masonry -- multiwythe or hollow unit masonry containing
12-3N M-4 (9-1-0) 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. <u>Only type M or S mortar shall be used</u>. Unreinforced masonry design procedures shall be used except that reinforced masonry areas or elements may be considered as resisting stresses in accordance with design criteria for reinforced masonry. <u>The width of these elements, tributary to the reinforcement, must meet the requirement of</u> effective width of masonry given in Section 12A.6. 3(A). Permissible shear stresses shall be determined in accordance with Section 12A.6.3(E). Permissible axial loads shall be determined in accordance with 12A.6.1. 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 12.2.1(B)3.

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
M-5 (9-0-1) 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 the 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.

5/3 cont.

M-5 cont.

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

These types of masonry walls shall be considered as reinforced masonry for the purpose of applying Table 12A-2. and section 12.4.1(G).

3. Masonry designed and reinforced with prescribed minimum areas in addition to the requirements of 12.2.1(B)(2). 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.

5/4

12-4N (10-0-0)

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

5/5

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.

12-4-1 (10-0-0)

~~(A) HEIGHT LIMITATION. Components of the seismic-resisting system in buildings under 35 feet in height shall be reinforced masonry when constructed using stacked bond and shall, as a minimum, be partially reinforced masonry when constructed using running bond. Components of the seismic resisting system in buildings over 35 feet in height shall be reinforced masonry and other structural components shall be partially reinforced masonry.~~

12-4-2/M-6 (10-0-0)

(A) DESIGN. Structural and nonstructural components of the building shall be designed and reinforced as specified in Table 12.1. The numbers designated 1, 2 and 3 in the Table refer to ~~Sections~~ subsections (1), (2) and (3) of ~~12.2.2.~~ 12.2.1(B).

5/6

(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.

12-4-3/M-7
(10-0-0)

5/6
cont.

(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 partially reinforced masonry designed as unreinforced masonry.~~

(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.

12-4-4
(10-0-0)

~~The maximum size of panels shall be 144 square feet with the maximum dimension in either direction of 15 feet. Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel. Supports at the top and ends of the panel shall be by means of confinement of the masonry by at least 1/2 inch into and between the flanges of a steel channel. The space between the end of the panel and the web of the channel shall be at least 1/2 inch and shall be void of mortar. The use of equivalent configuration in other steel sections or in masonry or concrete is acceptable.~~

~~Horizontal and vertical joints shall be not less than 1/4 inch thick. All joints shall be completely filled with mortar and shall be shoved joints.~~

5/7

(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. Cavity wall construction shall not be used for any structural masonry.

12-4-5 Committee voted to modify the table and move to seismic performance category C.

Table 12.2

(G) NOMINAL MINIMUM THICKNESS OF WALLS.

5/8 A B C D see below

TYPE OF MASONRY	Walls whose only structural function is exterior enclosure, nonstructural walls, and partitions	Thickness for the uppermost 35[2] foot high portion of wall
STRUCTURAL WALLS:		
Unburned Clay Masonry	16	16
Stone Masonry	16	16
Cavity Wall Masonry	8	12[3]
Hollow Unit Masonry	8	12[3]
Solid Masonry	8	12[3,4]
Grouted Masonry	6	10[3,4]
Reinforced Grouted Masonry	6	6
Reinforced Hollow Unit Masonry	~~4~~ 6[5]	6
NONSTRUCTURAL AND PARTITIONS:[1]		
~~Unreinforced~~	~~2~~	~~--~~
Reinforced	~~2~~ 4	--

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

[2]Seventy feet for stone masonry.

[3]These thicknesses may be reduced to 8 inches for walls that are not over 35 feet in total height in buildings that are not over three stories high.

[4]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.

~~5~~ 1 Except Nominal 4-inch-thick load-bearing 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 coverage where exposed to weather may be 1 1/2 inches.

$\boxed{\text{A - Modify as shown and move}}$ ← Committee preference

B - Modify as shown but keep in this location

C - Do not modify but move.

D - Do not modify and keep in this location

12-5

12-4-6/
M-13A
(9-1-0)

(G) MASONRY WALLS. ~~Masonry bearing wall thickness shall conform to (G) with a maximum h/t ratio of 25.~~

Except for walls designed under the provisions of Sections 12A.6.1 and 12A.6.2(A) ~~1. Stresses.~~ The axial stress in ~~reinforced~~ masonry bearing walls shall not exceed the value determined by the following formula:

5/9 M-13B (9-1-0)

$$f_m = 0.20\, f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right]$$

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 different from clear distance may be used if justified.

~~Other requirements are specified in 12A.6.3(E).~~

12-4-7
(10-0-0)

(H) REINFORCED MASONRY COLUMNS. Every structural wall ~~or pier~~ whose horizontal length is less than two times its thickness shall be designed and constructed as required for columns. The least dimension of every reinforced masonry column shall not be less than 12 inches and the maximum h/t ratio shall be 20.

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:

5/10

$$P = A_g\,(0.18\, f'_m + 0.65\, p_g\, f_s)\left[1 - \left(\frac{h}{40t}\right)^3\right]$$

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

p_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 = Clear height in inches.

Other requirements are specified in 12A.6.3(F).

12-4-8
(9-0-1)

(I) 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.

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 ~~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.

5/10
cont.

12-4-9/
M-16
(9-1-0)

(J) HIGH LIFT GROUTED CONSTRUCTION. For grouted masonry -multiwythe construction cleanouts shall be provided for each pour by leaving out every ~~other~~ third unit in the bottom tier of the section being poured. Other requirements are specified in 12A.3.4(B).

5/11

For hollow unit masonry construction cleanouts shall be provided for each pour by omitting face shells in the bottom course of each cell to be grouted. The grout ~~lift~~ pour shall not exceed 16 feet for walls 8 inches or more in nominal thickness nor ~~8~~ 12 feet for thinner walls. Cleaning shall be accomplished by means of a high-pressure jet stream of water, air jets, or other approved equivalent procedures. Other requirements are specified in 12A.3.6(B).

5/12

12-4-10
(8-2-0)

~~K (K) REQUIRED STRENGTHS FOR MORTAR AND GROUT. In addition to the requirements of Sec. 12A.8.2, minimum required strengths shall be 2000 psi for grout, 1500 psi for field mortar samples (2000 psi for field mortar cubes) unless higher strengths are required by the construction documents.~~

Note: Item (K) becomes Item (O) on p. 12-12 by a later ballot.

12-4-11 (10-0-0) (K) 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.

(L) GLASS MASONRY. Glass block shall be laid in Types
5/13 12-4-12 (10-0-0) 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.

(M) REINFORCEMENT DEVELOPMENT, ANCHORAGE AND SPLICES.
12-4-13 (10-0-0) The requirements of 12A.6.3(D) are applicable except that calculated
5/14 stress shall be replaced with yield strength. The following subsections 1 and 2 replace subsections 5 and 7, respectively, in 12A.6.3(D).

5/14 cont.

12-4-14 (10-0-0)

1. Development lengths. The basic development length, l_d, for deformed reinforcement shall be at least $.05\ d_b\ f_y/\sqrt{f_g}$ but not less than 24 d_b for reinforcement of 40,000 psi yield strength nor $36d_b$ 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 conf[illegible] 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 system, these values may be reduced to $18d_b$ for bars of 40,000 psi yield strength and $27d_b$ 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.

12-4-15 (10-0-0)

2. 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 $0.08\ d_b\ f_y/\sqrt{f_g}$ but not less than $40d_b$ 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. 134.6.3(1)5.

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

12-4-16
(9-0-1)

N
(~~P~~) 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.

5/15

12.4.2 MATERIAL LIMITATIONS.

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

Unburned Clay Masonry

Structural Clay Load Bearing Tile

12-4-17/
M-18
(9-1-0)

Mortar with Air Contents Greater than 15%
Masonry Cement ~~(Mortar and Grout)~~

~~Mortars other than types M or S~~.

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.

12-5N/
M-19
(10-0-0)

(A) REINFORCEMENT. All masonry shall be reinforced masonry conforming to section 12.2.1(B) except for one-story residences of running bond construction located in map area 5 shall conform to section 12.2.1(B)2.

(B) TIE ANCHORAGES. In addition to the requirements of Sec. 12A.6.3(C), 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.

5/16

(C) REINFORCED COLUMNS. In addition to the requirements of Sec. 12.6.3.5 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 16 inches.

(D) SHEAR WALL BOUNDARY ELEMENTS. Boundary members 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.

12-6-N
(10-0-0)

(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.

12-6-1/ M-20 (10-0-0)

WALLS
(G) ~~PIERS~~. Every structural wall ~~or pier~~ in reinforced masonry construction whose horizontal length is between ~~3~~ 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.

5/17 12-6-2/ M-21 (10-0-0)

(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.~~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 Regulatory 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.

5/18 12-6-3 (8-1-1)

(I) BOLT PLACEMENT. Bolts shall be accurately set ~~with templates or by approved equivalent means~~ and held in place to prevent movement.

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.5.3(F) and 12.4.1(B) for bolts at the top of ~~piers~~, pilasters and columns.

5/19 12-6-4 (5-2-3)

~~(J) 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.~~

Notes: Item I, unchanged, also becomes Item D on p. 12-14. Item J becomes Item E on p. 12-14.

5/20	12-6-5 (8-1-1)	(K) GROUT. 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~~. Mixing equipment and procedures shall produce grout with the uniformity required for concrete by ASTM C94.
	12-6-6/ M-22 (10-0-0)	(L) ALLOWABLE STRESSES WITHOUT SPECIAL INSPECTION. The allowable stresses for ~~uninspected reinforced construction~~ work without special inspection shall be those given in Table 12A.5 except that the factor of 2/3 for axial compression in walls and columns shall be reduced to 1/2.
5/21	12-6-7/ M-24 (10-0-0)	(M) CORE TESTS FOR SHEAR BOND IN GROUTED MASONRY-MULTIWYTHE. In addition to the requirements of Sec. 12A.3.3 the following provisions must be met for ~~all~~ 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.
5/24	M-32 (10-0-0)	(N) MASONRY WALLS. Masonry wall thickness shall conform to table 12-2. The ratio of height or length to thickness of reinforced structural walls shall not exceed 25.
	M-33 (10-0-0)	(O) Insert Item (K) from p. 12-7.

12.5.2 MATERIAL LIMITATIONS.

5/22		The following materials shall not be used for any structural purpose:
	(3-2-0)	~~Building Brick and Hollow Brick made from Clay or Shale of Grade NW~~
	(10-0-0)	~~Concrete Building Brick and Solid Load-bearing Concrete Masonry Units other than Grade N~~
5/23	(10-0-0)	Hollow Load-bearing Concrete Masonry Units other than Grade N
	(7-3-0)	~~Sand-lime Building Brick other than grades SW and MW~~
	(7-3-0)	Type N Mortar
	(7-3-0)	Masonry Cement

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

(10-0-0) ~~Glass Units~~

5/25 Unburned Clay Masonry

Structural Clay Load-bearing and Nonload-bearing Wall Tile

(9-0-1) Masonry Cement (Mortar with Air Content Greater than 15%)

(10-0-0) ~~Mortar Types N, O and K~~

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.

5/26 (B) REINFORCED HOLLOW UNIT MASONRY. Structural reinforced hollow unit masonry shall conform to requirements below:

1. Wythes and elements shall be at least 3 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 10 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 192 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.5.3(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 structural 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.

5/26 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.

(10-0-0) (D) Insert Item (I) from p. 12-11 reinserting the phrase "with templates or by approved equivalent means."

(7-1-2) (E) Insert Item (J) from p. 12-11.

(10-0-0) (F) Insert Item (K) from p. 12-12 reinserting the phrase "not more than 10 inches for fine grout or 9 inches for coarse grout."

12.5.2 MATERIAL LIMITATIONS

12-8-1 M-28 (10-0-0)

~~other than Grade N,~~

Hollow nonload-bearing concrete masonry units shall not be used. Sand-lime building brick, ~~shall not be used for any structural masonry.~~ Building Brick and Hollow Brick made from Clay or Shale of Grade NW and Building Brick and Solid Load-bearing Concrete Masonry Units other than Grade N shall not be used for any structural masonry.

12.5.3 SPECIAL INSPECTION

Special inspection shall be provided for all structural masonry.

12.7 <u>SHEAR WALL REQUIREMENTS</u>

Shear walls shall comply with the requirements of this Section.

12.7.1 REINFORCEMENT

12-9 (10-0-0)

5/27 The following reinforcement requirements apply to shear walls required to comply with the provisions of 12.2.1(B)(3).

12-10 (10-0-0) 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.

5/27 cont.

12-10-1 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.

(10-0-0)

M-29 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.

(10-0-0)

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.

12.7.2 BOUNDARY MEMBERS

5/27 cont.

Where cross walls or boundary members form a part of the shear wall system, the intersections 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 3-8, 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.C, for Category C & 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.

M-30 (10-0-0)

12.7.3 ~~COMPRESSIVE~~ 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 stresses 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-7 shall not be used.

5/27

12-11
(10-0-0)

EXCEPTION:
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-7 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 ~~true~~ joints. The joints shall provide for building movement determined in accordance with Sec. 3.3. The horizontal components shall be anchored to the building and designed as otherwise required by these provisions.

5/28

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	1	2'	2	2	2	3
Stacked Bond	2	2	3	3	3	3
Nonstructural Components						
Running Bond	1	1	1	1	2	2
Stacked Bond	1	1	1	2	2	2

Note: 1. The numbers 1, 2 and 3 refer to subsections (1), (2) and (3) of 12.2.1(B)

2. Map areas refer to figure 1.2.

12-11-1
(9-0-0)
1 Abstain

Chapter 12 ballots are based on an ATC proposed (working with committee members) draft that reflects the results of Chapter 12A ballots. The draft moves forward seismic items deleted from Chapter 12A and revises wording for compatiblity.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/1	• Partial deletion and insertion - editorial change.	Yes - Editorial changes.
5/2	• New insert - editorial change for clarification. • New insert - seismic requirement from Ch. 12A	Yes - Editorial change and one of the items brought forward from Ch. 12A.
5/3	• Deletion and Insertion - The section has been rewritten to be compatible with the approach in Ch. 12A in omitting the terminology "partial reinforcement." Care was taken in (B) Reinforced Masonry, to insure that sufficient reinforcement was present before permissible stresses for reinforced masonry could be used. Research is necessary to determine whether or not lower limits are possible. Reinforcement spacing and detailing have been moved from Chapter 12A.	Yes - These three definitions for reinforced masonry have been added because of Committee 5's desire to remove the terminology "partial reinforcement" from the document. They wanted to refer to reinforced masonry as masonry containing reinforcement regardless of its amount. This proposed ballot item certainly attains this objective but may lead to a more confusing situation with regard to what allowable stresses can be used when reinforcement is present. The allowable stresses for reinforced masonry have been developed on the assumption of prescribed minimum amounts. The wording in 12.2.1 (B) 2 attempts to clarify what allowable stresses can be used for the previously used "partial reinforcement" requirements. ATC recommends that this section be editorially revised.
5/4	• No change - The committee preferred reference to national standards but this was not completely possible.	Yes - No change from text.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/5	• Deletion and insertion - The proposed ATC wording was accepted with editorial change. The referenced Table 12-1 was modified in Ballot Item 5/28.	Yes - This comment pertains to Sec. 12.4.1 (A). This is a major change from the original requirements of Ch. 12.4.1 (A) and may produce an inconsistency with Category B requirements for Reinforced Concrete Ordinary Moment Frames. ATC recommends that the change from 2 to 1 for structural components in Table 12-1 with running bond in buildings over 35 feet high not be approved. Furthermore, the changes inherent in this table should be reviewed and commented upon by Committee 2 for consistency with other material requirements. However, ATC proposed this change of refinement in SPC-B.
5/6	• Deletion and insertion - This is the original text of 12.4.1 B and D. Part C is a minor change to delete reference to "partial reinforcement."	Yes - Pertains to 12.4.1 (B), (C), (D). Primarily original text.
5/7	• No change - original text.	Yes - Pertains to 12.4.1 (E), (F). Original text.
5/8	• Transfer, insertion, partial deletion - The table was felt to be unnecessary in SPC-B. Thickness would be controlled by the limiting h/t restrictions and available masonry units. The deleted types of structural walls are not allowed and therefore not needed. The 35-foot restriction is not needed and is deleted. Deletion of footnotes follow deletion of above items.	Option D - We believe this item should be in Ch. 12A but if removed from 12A, it is recommended to be retained under SPC-B.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/9	• Transfer and deletion - The section on masonry walls was transferred from Ch. 12A by the ATC rewrite. The lower coefficient of 0.20 for seismic rather than the 0.225 used in Ch. 12A was considered satisfactory. The h/t restriction was considered unjustified. Data are available for wall tests using h/t of up to 40. The h/t = 25 limit is in SPC-D as 5/24. The committee agrees with the ATC interpretation of a "no" vote.	No - This item has been transferred from Ch. 12A. ATC believes that the h/t limitation that is proposed for deletion should be retained, and, therefore, the ballot item not be passed with h/t deleted. A "no" vote means retention of the section including the h/t restriction. "Yes" means as shown.
5/10	• Transfer - The transfer from Ch. 12A with the lower coefficient of 0.18 for seismic rather than the 0.20 used in Ch. 12A was considered satisfactory. The transfer of (I) from Ch. 12A is supported.	Yes - Pertains to 12.4.1 (H), (I). Items brought forward from Ch. 12A.
5/11	• Transfer - The transfer of (J) from Ch. 12A for seismic is considered satisfactory.	No - Pertains to 12.4.1 (J). Recommend retention of leaving out every other unit for cleanouts.
5/12	• Transfer - This item was considered unnecessary in Ch. 12A since ASTM specifications provide the needed control. The same rationale holds for SPC-B. Its inclusion in SPC-C is supported (see Ballot Item 5/24).	No - This ballot item is identical to Ballot Item 5A/72 except that the committee proposes to move it from SPC-B to SPC-C. This item does not really relate to SPC because it is a quantitative quality control item for mortar and grout. Ifdeleted from Ch. 12, ATC strongly believes that it should remain in SPC-B and, therefore, recommends that this item not be passed.
5/13	• Transfer - The committee supports the transfer from Ch. 12A to Ch. 12.	Yes - Items (K) and (L) are just items brought forward from Ch. 12A.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/14	• Transfer - Ch. 12A is based on allowable stresses and includes material similar to this. Ch. 12 is based on strength. The committee supports transfer of this strength-related material to Ch. 12.	Yes - This item was brought forward from Ch. 12A and includes the development length for the yield stress of the reinforcement.
5/15	• Transfer, modification - Section (N) was transferred to Ch. 12 for seismic. The other changes are minor.	Yes - Pertains to 12.4.1 (N) and 12.4.2. Item (N) was brought forward from Ch. 12A. No objection to other change.
5/16	• Insert - These are the original SPC-C requirements except for changes in (A) and (E). (E) is editorial. Item (A) insert relaxes the reinforced masonry requirements for single-story residences.	Yes - Pertains to 12.5.1 (A), (B), (C), (D), (E), (F). This was the original SPC-C requirements including a liberalization for one-story masonry residences.
5/17	• Transfer - These are items brought forward, with minor change, from Ch. 12A.	Yes - Pertains to 12.5.1 (G), (H). Items were brought forward from Ch. 12A.
5/18	• Transfer - item (I) was transferred from Ch. 12A for seismic reasons. The modified wording adopted for SPC-C is performance based with the requirement being to accurately set the bolts. Use of only templates is too restrictive. It should be noted that (I) with templates inserted back in is supported in SPC-D by 5/26 as item D with its generally more restrictive requirements.	No - Recommend retention of templates or equivalent means and that this item be retained in SPC-C.

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/19	• Transfer - This item was deleted in Ch. 12A but reconsidered for Ch. 12. Shrinkage was considered a serviceability and not a safety or seismic problem. It was adopted in SPC-D by 5/26 as item E.	No - Pertains to 12.5.1 (J). It was recommended that the shrinkage limits be retained in Ch. 12; however, if not, then it should occur in SPC-C.
5/20	• Transfer, deletion - The item was transferred from Ch. 12A. See remarks on 5A/16.	No - Pertains to 12.5.1 (K). Same reason as given in 5A/16.
5/21	• Transfer - Transfer from Ch. 12A is supported.	Yes - Pertains to 12.5.1 (L), (M), brought forward from Ch. 12A.
5/22	• Deletion - The NW grade is for durability purposes. It does not affect strength so there is no need for the restriction.	No - First listed item under 12.5.2 is recommended to be retained.
5/23	• Partial deletion and inserts - The changes are considered more appropriate for the specific materials shown.	Yes - No objections to remaining changes or additions made in Material Limitations list.
5/24	• Transfer - Item N was moved from SPC-B (5/9) to SPC-D. Item O was moved from SPC-B (5/12).	Yes - Also recommended that this be retained under SPC-B.
5/25	• Partial deletion, replacement - More appropriate use of materials.	Yes - No objection to changes and additions made under "nonstructural materials list."

Joint Committee Ballot No.	Type of Change and Committee Comment	ATC Recommendation and Comment
5/26	• No change - This is Sec. 12.6 of Ch. 12 in its entirety.	Yes - Pertains to 12.6 in its entirety.
5/27	• Transfer - Items (D), (E), (F) are similar to ballot items 5/18, 5/19, and 5/20, respectively. Other changes are minor.	Yes - No objection to remainder of changes and addition.
5/28	• Partial deletion and insertion - The table was proposed by ATC. The ballot refers only to changing the 2 to a 1 for Map Area 2 for buildings over 35 feet. The table and footnotes were accepted, otherwise.	No, with explanation - A "no" vote infers the retention of 2 rather than 1 for buildings over 35 feet in Map Area 2.

COMMITTEE 6

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6 - Steel

COMMITTEE ITEM NUMBER:

ATC-3-06 SECTION REFERENCE: 10.2

Change the seventh and eighth line to read as follows:

"...members or structural systems."

"Connections which do not develop the strength of the member or structural systems..."

FINAL BALLOT: 5 YES
0 NO
0 ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

These changes reflect the cases where members need not develop the full capacity of their cross section.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6, Steel

COMMITTEE ITEM NUMBER: ____

ATC-3-06 SECTION REFERENCE: 10.2.1

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

FINAL BALLOT: 5 YES
0 NO
0 ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Deletion of sec. 10.2.1 (B) is to prevent the use of Eq. (2.5-1) of Part 2, AISC Specs. which limits the maximum allowable shear to (0.55 F_y) td. The committee recommends that the maximum allowable shear be increased to (0.68 F_y) td.
Addition of sec. 10.2.1 (D) reflects this recommendation.
This is also to be consistant with the proposed AISC specifications.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6, Steel

COMMITTEE ITEM NUMBER: ____

ATC-3-06 SECTION REFERENCE: 10.4

Delete sec. 10.4.1
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."

FINAL BALLOT:
5 YES
0 NO
0 ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Combine two separately stated requirements into one.

APPENDIX B

INSTRUCTIONS FOR THE JOINT COMMITTEE BALLOT

The Joint Committee ballot was conducted in two stages. The instructions for these two ballots were conveyed in two letters from the chairman, E. O. Pfrang, dated July 18 and July 24, 1980. These two letters and their attachments (exclusive of the actual ballot sheets and ballot items) are included in this appendix.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6, Steel

COMMITTEE ITEM NUMBER: ______

ATC-3-06 SECTION REFERENCE: 10.5.1

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 building assigned to Seismic Performance Category D may be Ordinary Moment Frames.

FINAL BALLOT:
5 YES
0 NO
0 ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

One-story steel frame buildings have performed well during earthquakes. Addition of "Exception 2" reflects these case histories, and the intent of sec. 3.3.4 and 3.3.5.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6, Steel

COMMITTEE ITEM NUMBER: ______

ATC-3-06 SECTION REFERENCE: 10.6.3

Change the third line to read "axial force in the columns shall not exceed 0.75 P_y."

FINAL BALLOT:
- 3 YES
- 2 NO
- 0 ABSTAIN
- 1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Modifying the 0.6 factor to 0.75 was to reflect the change in the AISC specifications (1979). Since the 1979 AISC specs. has already incorporated the 0.75 factor, this item should be deleted (Pinkham's letter of April 24, 1980). It has been suggested that this factor be kept as 0.6 and evaluate its impact by trial designs.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #6, Steel

COMMITTEE ITEM NUMBER: ______

ATC-3-06 SECTION REFERENCE: 10.6.5

Change the first line after the equation to read as follows:

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

FINAL BALLOT:
5 YES
0 NO
0 ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The committee felt that the present equation is too complicated, and a simpler form is desired. However, the committee agreed to retain the equation in its present form, and examine its impact during the trial design phase. Because the way in which the equation was derived, the equation should replace only AISC Eq. 1.15-2 which is concerned with bucking of the column web.

COMMITTEE 7

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 1

ATC-3-06 SECTION REFERENCE: 9.1

Add new references:

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

FINAL BALLOT: 5 YES
____ NO
____ ABSTAIN
____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Plywood working stresses are included in the Plywood Design Specification. It will be necessary in some cases to check the shear strength of plywood in order to design a plywood diaphragm by the principles of mechanics. This information is not contained in any of the other references in Chapter 9.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood   COMMITTEE ITEM NUMBER: 2

ATC-3-06 SECTION REFERENCE: 9.1

Change Reference 9.12 to read:

One- and Two-Family Dwelling Code, 1975*

*One of the affirmative voters made an editorial note that the latest edition of this code is 1979.

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

As written in 9.12, it appears as though each of the three model codes has a one- and two-family code. The One- and Two-Family Dwelling Code is a single document written by the four model code organizations, the three listed in 9.12 plus the American Insurance Association.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 3(A)

ATC-3-06 SECTION REFERENCE: 9.2

Change the capacity reduction factor, ϕ, for shear on diaphragms and shear walls, from 0.75 to 0.85.

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The change was suggested on the basis of the results of a diaphragm test program conducted by the American Plywood Association. It was found that the average load factor against failure was 3.65, which exceeds the product of the multiplying factor, 2, (see Section 9.2) times 0.85 by more than 2. In light of this comparison, the Committee agreed to increase the value of ϕ from 0;75 to 0.85.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 3(3)

ATC-3-06 SECTION REFERENCE: 9.2

Revise the tabulation of strength reduction factors 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	$\phi = 0.85$

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The change deletes the ϕ values for nails in shear in plywood diaphragms of Group III species members ($\phi = 0.82$) and Group IV species members ($\phi = 0.65$) because these values are included in Tables 9-1 and 9-2 of ATC-3-06. The ϕ value for shear on diaphragms and shear walls was changed from 0.75 to 0.85 per Committee Item No. 3(A). The two previous values of ϕ (0.90 & 3.6/N) for lag screws and wood screws were changed to a single value of $\phi = 0.90$.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood   COMMITTEE ITEM NUMBER: 4

ATC-3-06 SECTION REFERENCE: 9.4.1(c)

Delete this subsection.

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This change was approved in view of the fact that the 1977 edition of the National Design Specification (Reference 9.1) has covered this requirement.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 5

ATC-3-06 SECTION REFERENCE: 9.5.3(A)

Replace the existing language with the following: Reference 9.1 shall be modified as follows: In 8.8.1.4, replace the existing language with "When more than one nail or spike is used in a joint of a frame or similar component, the total design value shall be determined in the same manner as is done in 8.3.2.3." In 8.8.6, change two-thirds to one-half.

FINAL BALLOT: 4 YES (1 with comment)
1 NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The opinions of the negative voter were that: 1) Section 8.3.2.3 of the National Design Specification (NDS) does not apply to nails; thus the proposed change is inappropriate and is not substantiated, and 2) there is no justification for the suggested change in NDS 8.8.6 since the two-thirds factor has been in the NDS since its inception. The affirmative voter that had comments was of the opinion that Committee 7 should be hesitant to reference the NDS and then to suggest changes in the NDS. He also felt that the modifications suggested for the NDS were based on "gut" feelings rather than fact.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 6

ATC-3-06 SECTION REFERENCE: 9.5.3(B)

Remove this subsection and transfer it to Section 9.6.3.

FINAL BALLOT: 5 YES
____ NO
____ ABSTAIN
____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The Committee agreed with the imposition of special requirements in Category D construction in so far as plywood over gypsum sheathing is concerned. It was felt that prohibiting the use of gypsum sheathing as a part of the seismic resisting system was not justified for Category C construction. This opinion was based on the results of some shear tests on walls using plywood applied over gypsum wallboard. The average load factor obtained in the testing program was greater than 4.5.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 7

ATC-3-06 SECTION REFERENCE: 9.6.3

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

FINAL BALLOT: 5 YES
____ NO
____ ABSTAIN
____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 8

ATC-3-06 SECTION REFERENCE: 9.7.1(A)

Section 9.7.1 can be modified as follows: ...provided at not over 6 feet on center for buildings two stories, 20 feet, or less in height and at not over 4 feet on center for buildings over this height but three stories, or 35 feet or less in height. Anchor bolts shall have a minimum embedment of 8 diameters.

FINAL BALLOT: 3 YES
2 NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The proposed change was predicated on the fact that the Uniform Building Code has permitted anchor bolt spacing of 6 feet for many years with no documented detrimental consequences in recent earthquakes including the 1971 San Fernando, California Earthquake. One minority view was that in the Great Alaska Earthquake there were some sill/foundation anchorage failures caused by undefined forces. Until we know what kind of forces are acting, the anchorage requirements should be more, rather than less, conservative. A second view was that bolts should not be less than 5/8 in diameter at 4'0" on center with at least 7 in embedment. The need for strengthening the anchorage provision was also suggested by a member of Technical Committee #9.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood

COMMITTEE ITEM NUMBER: 9

ATC-3-06 SECTION REFERENCE: 9.7.1(c)

Delete the word "stud" at the end of the sentence and add the following "...studs unless specifically excepted in Section 9.7.3."

FINAL BALLOT: 4 YES
___ NO
1 ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 10

ATC-3-06 SECTION REFERENCE: 9.7.3(B)

Add the sentence: "Blocking need not be provided at horizontal joints."

FINAL BALLOT: 4 YES
NO
1 ABSTAIN
DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

It was the view of the proponent of this change that for conventional light-timber construction it is not necessary to block horizontal joints in plywood sheathing. The primary basis for this opinion was the results of four tests on walls which were sheathed with 5/16" cedar panels. The minimum ultimate load obtained for these Group IV species (i.e. the lowest strength group recognized for sheathing applications) was 4400 lb. It is implied that this magnitude is sufficiently high to preclude the failure of the plywood bracing panels.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 11

ATC-3-06 SECTION REFERENCE: Table 9-1

- Change the table heading to read: ALLOWABLE SHEAR IN POUNDS PER 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 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 "BLOCK 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).

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This set of editorial changes is necessary to make Table 9-1 agree with Table No. 25-J of the 1979 Edition of the Uniform Building Code.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 12

ATC-3-06 SECTION REFERENCE: Table 9-2

Revise the table as shown on the attached sheet.

FINAL BALLOT: 5 YES
___ NO
___ ABSTAIN
___ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The table was updated to agree with Table No. 25-K of the 1979 Edition of the Uniform Building Code. This is primarily an editorial change involving the re-arrangement of columsn with no changes in the numbers in the table. The previously omitted allowable shear value (200) for siding attached with 8d nails at 4 inches on centers was inserted.

TABLE NO. 9-2 —ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES IN POUNDS PER FOOT FOR PLYWOOD SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[1]

PLYWOOD GRADE	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED DIRECT TO FRAMING — Nail Spacing at Plywood Panel Edges: 6	4	2½	2	NAIL SIZE (Common or Galvanized Box)	PLYWOOD APPLIED OVER ½ INCH GYPSUM SHEATHING — Nail Spacing at Plywood Panel Edges: 6	4	2½	2
STRUCTURAL I	5/16	1¼	6d	200	300	450	510	8d	200	300	450	510
	3/8	1½	8d	230[3]	360[3]	530[3]	610[3]	10d	280	430	640[3]	730[3]
	1/2	1⅝	10d	340	510	770[3]	870[3]	—	—	—	—	—
C-D, C-C, STRUCTURAL II and other grades covered in U.B.C. Standard No. 25-9	5/16[2]	1¼	6d	180	270	400	450	8d	180	270	400	450
	3/8	1½	8d	220[3]	320[3]	470[3]	530[3]	10d	260	380	570[3]	640[3]
	1/2	1⅝	10d	310	460	690[3]	770[3]	—	—	—	—	—
			NAIL SIZE (Galvanized Casing)					NAIL SIZE (Galvanized Casing)				
Plywood Panel Siding in Grades Covered in U.B.C. Standard No. 25-9	5/16	1¼	6d	140	210	320	360	8d	140	210	320	360
	3/8	1½	8d	130[3]	200[3]	300[3]	340[3]	10d	160	240	410	410

[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 No. 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.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 13

ATC-3-06 SECTION REFERENCE: 1.3.1

Modify the last line to read "conventional light timber construction as permitted in Section 9.5."

FINAL BALLOT:
3 YES
1 NO
ABSTAIN
1 DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The revision subjects one- and two-story wood frame dwellings, not over 35 feet in height and located in areas having Seismicity Index 3 or 4, to the requirements of Seismic Performance Category C (Section 9.5). As was indicated by the person not voting, the intent of the change is not clear. The negative voter was strongly opposed to this change in that he felt the provisions of Section 9.7 as presently required are more than adequate to assure a safe building.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

PROPOSED CHANGE

TECHNICAL COMMITTEE: #7, Wood COMMITTEE ITEM NUMBER: 14

ATC-3-06 SECTION REFERENCE: 14.6

Add to the reference documents: 1) Plywood Design Specification, 1978, APA and 2) Plywood Diaphragm Construction 1978, APA.

FINAL BALLOT: 5 YES
____ NO
____ ABSTAIN
____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This is an editorial change.

COMMITTEE 8

2.0 Committee Actions

2.1 Recommendations for Change

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL   COMMITTEE ITEM NUMBER: 1/1

ATC-3-06 SECTION REFERENCE: 8.2.5

First Letter Ballot
Item 1

°Section 8.2.5 to read: "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"

FINAL BALLOT: 8 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The change is suggested as a more performance approach to cover a broad range of materials.

TECHNICAL COMMITTEE: #8 ARCHITECTURAL, MECHANICAL & ELECTRICAL

COMMITTEE ITEM NUMBER: 1/2

ATC-3-06 SECTION REFERENCE: Commentary 8.2.5

First Letter Ballot
Item 2

°Commentary 8.2.5, third sentence to read: "This is particularly important for systems composed of brittle materials and/or low flexural strength materials."

FINAL BALLOT:
- 8 YES
- --- NO
- --- ABSTAIN
- --- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This change is editorial and was made for consistency.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL   COMMITTEE ITEM NUMBER: 1/3

ATC-3-06 SECTION REFERNCE: TABLE 8-B

First Letter Ballot
Item 3

°Table 8-B Footnote 4 changed 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."

FINAL BALLOT:
8 YES
--- NO
--- ABSTAIN
--- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Footnote 4 changed to be more specific with regard to appendages and accessibility to areas near the appendages.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL

COMMITTEE ITEM NUMBER: 1/4

ATC-3-06 SECTION REFERENCE: TABLE 8-B

First Letter Ballot
Item 4

°Table 8-B: Change entry "Veneers" to "Veneer Attachments"

FINAL BALLOT:
- 8 YES
- --- NO
- --- ABSTAIN
- --- DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Editorial change to better reflect the intent of the entry in Table 8-B which is the "attachments" rather than the vaneer.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL   COMMITTEE ITEM NO.: 2/1

ATC-3-06 SECTION REFERENCE: 8.1

Seond Letter Ballot
Ballot Item 1

(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.

FINAL BALLOT: _____ YES
_____ NO
_____ ABSTAIN
_____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

The additional "exception" was appropriately added to Section 8.1 so that the exceptions would be in one place. This new exception could have been placed in the new Section 8.4 but it was decided the better location would be in Section 8.1.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL, MECHANICAL & ELECTRICAL   COMMITTEE ITEM NUMBER: 2/2

ATC-3-06 SECTION REFERENCE: 8.4 (New Section)

Second Letter Ballot
Ballot Item 2

(New Section to be added to Chapter 8)

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 ELEVATOR 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 weight of 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.

BALLOT ITEM 2 CONTINUED

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 electical alert or command for the safe automatic emergency operation of the elevator system.

(b) A counter weight 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 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.

FINAL BALLOT: _____YES
_____NO
_____ABSTAIN
_____DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

Because of the significance of elevator performance and potential cost impact, it was decided that a new section should be developed specific to "Elevator Design Requirements." The material has been developed so that the existing formulas and proposed seismic coefficients can be used in designing to resist seismic forces.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL   COMMITTEE ITEM NUMBER: 2/3

ATC-3-06 SECTION REFERNCE: TABLE 8-B

Second Letter Ballot
Ballot Item 3

1. Change entry under Partitions - "Elevators and Shafts" to Elevator Shafts. (Editorial)

2. Add the following new entry:

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

FINAL BALLOT: _____ YES
_____ NO
_____ ABSTAIN
_____ DID NOT VOTE

COMMENTS ON PROPOSED CHANGE:

These new entries to Table 8-B allow general usage of equation 8-1.

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS PROPOSED CHANGE

TECHNICAL COMMITTEE: #8 ARCHITECTURAL MECHANICAL & ELECTRICAL

COMMITTEE ITEM NUMBER: 2/4

ATC-3-06 SECTION REFERENCE: TABLE 8-C

Second Letter Ballot
Ballot Item 4

1. Add the following new entry:

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

FINAL BALLOT: _____ YES
_____ NO
_____ ABSTAIN
_____ DID NOT VOTE

COMMENT ON PROPOSED CHANGE:

This new entry to Table 8-C allows the general usage of equation 8-2.

COMMITTEE 9

JOINT BALLOT NUMBER

2.0 Committee Actions

2.1 Recommended Changes

The following is a compilation of the results of the Committee 9 ballot issued on April 14, 1980.

JOINT BALLOT NUMBER	Section	Affirmative	Negative	Affirmative with Reservations	Did Not Vote
9/1	° Chapter 1 - Change of title for Chapter 1 from "Administration" to "General Provisions"	6 votes	2 votes[1/][2/]	-0-	4
9/2	° Section 13.1.1 - Change the word "designed" to "with a permit issuance date" in paragraphs one and two of Section 13.1.1.	7 votes	-0-	1 vote[3/]	4

Summary of "Remarks" offered on negative and affirmative with reservations ballots:

1/ "The chapter should be titled 'General'. This document is not a code - 'General Provisions' is code language".

2/ "Propose 'Application'".

3/ "With regard to seismicity index of 4, Add asterisk and Note: Local jurisdiction may change this for their location".

2.2 Recommendations for Trial Designs

1. The committee recommends that economic studies be undertaken in conjunction with the trial designs to determine the economic impact of the provisions on one- and two-family dwellings in all Seismicity Zones.

2. It was recommended that the seismicity index numbers appear on the map legends to correspond to the designated map areas.

3. Chapter 13 should be re-written so that it can be comprehended by the layman. As presented in its current version, the chapter is difficult to understand and comprehend.

4. Chapter 13 should be reviewed for its applicability to the Eastern part of the United States. R_c of 1 is for all practical purposes not attainable for most older buildings in the Eastern part of the U.S.

COORDINATING COMMITTEE

Review and Refinement of Tentative Seismic Provisions

Proposed Changes

Coordinating Committee

Joint Ballot Number	Item
S/1	Table 1-B: change the Seismicity Index for Map Area 5 from 4 to 3.
S/2	Table 1-B: change the Seismicity Index for Map Area 4 from 3 to 2.
S/3	Table 1-B: change the Seismicity Index for Map Area 3 from 2 to 1.
S/4	Table 1-B: change the Seismicity Index for Map Area 2 from 2 to 1
S/5	Table 1-B: change the Seismicity Index for Map Area 1 from 1 to 0.

Comment:

Several of the technical committees balloted various changes of the seismicity index. Because of the intense interest, the Coordinating Committee decided to pull all these items together for the Joint Committee ballot.

Review and Refinement of Tentative Seismic Provisions

Proposed Changes

Coordinating Committee

Joint Ballot Number	Item
S/6	Revise Joint ballot number 3/4, as follows: At the end of paragraph 2 of Section 7.4.4 (before Item A) the following sentence should be added, "Where special reinforcement at the top of the pile is required, alternative measures 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."
S/7	Revise joint ballot number 4/5 as follows: Add the following sentence at the end of paragraph 7.4.4(E): "Pile cap connection may be by means of developing exposed pile reinforcing strand if a ductile connection is provided."
S/8	Add the following sentence to the end of section 7.5.3(C): "Pile cap connection shall <u>not</u> be made by developing exposed strand."

Comment:

These revised ballot items resolve conflicts between changes proposed by Committees 3 and 4.

UNITED STATES DEPARTMENT OF COMMERCE
National Bureau of Standards
Washington, D.C. 20234

July 18, 1980

MEMORANDUM TO: Participants in the Review and Refinement of the Tentative Seismic Provisions

Dear Participant,

For those who did not attend the meeting on July 16-17 at NBS, a copy of the proposals for change is enclosed, along with a ballot to be returned to NBS, postmarked no later than July 31, 1980. An addressed and franked envelope is enclosed for your use. Please note: if you were represented at the meeting by a proxy, this material was given to your proxy.

For all participants, some additional information is enclosed as follows:

- a statement from Committee 4 concerning ballot item 4/12 (four pages)
- a statement from Committee 8 concerning ballot item 8/6
- ATC comment on ballot item 4/12 (two pages)
- ATC presentation on Committee 1 by Neville Donovan (three pages)
- information summarizing the actions of various Committees on proposals that are the same, or similar to, ballot items proposed by different committees. These items are 4/3, 4/6, 4/8, 4/9, and S/1 through S/5 (three pages).

In connection with the last item, please note ballot item 2/10 when you consider ballot item 4/11.

All participants should note: the ballot items proposed by Committee 5 (5/1-13 and 5/A1-A37) should not be balloted at this time. The Committee has subdivided their ballot, and they are preparing documentation for each ballot item. This is with intent of facilitating your balloting of their proposals. The revised material and supplementary ballot will be sent to you in one week and will be due one week after the basic ballot. This supplementary ballot will also include proposals from Committee 4, but you are requested to completely ballot Committee 4's proposals on the basic ballot and then consider their supplementary proposals in an independent ballot.

Please note that the ballot is a "YES/NO" ballot. This does not mean that you may not comment. You are encouraged to send comments along with your ballot. They will be summarized for the BSSC.

If you have any questions regarding these items, please call Dr. J. R. Harris at (301) 921-2170.

Sincerely,

E. O. Pfrang, Chairman
Joint Committee on Review and Refinement

COMMITTEE 4 - CONCRETE

BALLOT ISSUE 4/12 REPORT

1. Committee 4 met four times. From the second meeting onwards the major issue became a proposal to adopt the latest proposed revision to ACI 318 Appendix A in place of the existing Chapter 11 of ATC 3-06. Initially, the Committee was reluctant to take that step because of anticipated difficulties in integrating Appendix A with the remainder of ATC 3-06 and with other proposals developed by the Committee for revisions to the existing Chapter 11 of ATC 3-06. However, through the dedicated effort of several Committee members, and particularly the PCA representative, a comprehensive proposal, ballot item 4/12, was developed that:

(a) provides a substitute Chapter 11 incorporating the 19 March 1980 proposed revision to Appendix A of ACI 318-77,

(b) contains provisions that interface Appendix A with the remainder of ATC 3-06 while simultaneously incorporating other proposals accepted by the Committee as desirable revisions to the existing Chapter 11, and

(c) identifies the changes necessary elsewhere in ATC 3-06 for incorporation of the substitute Chapter 11.

2. In its deliberations the Committee made an in-depth review of Chapter 11, compared its design provisions with those of the proposed revision to Appendix A of ACI 318-77 and realized that numerous changes would be necessary to upgrade existing Chapter 11 to the latest ACI criteria. Thus, the majority of Committee 4 is recommending the adoption of the new ACI provisions, considering this to be the most efficient approach and in the best national interest for trial designs.

Therefore, with respect to Chapter 11, Committee 4 is recommending in ballot issue 4/12 that the nationally accepted design standard ACI 318-77 "Building Code Requirements for Reinforced Concrete," including proposed revision - Appendix A "Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions," dated 19 March 1980, be adopted by reference into ATC 3-06 for proportioning and detailing concrete structures. Revised Appendix A is now before the full ACI Building Code Committee 318. Final Committee action and full ACI consensus balloting is forthcoming.

3. The vote on ballot issue 4/12 by Committee 4 was 7 in favor and 1 opposed.

4. The reasons advanced by the majority of the Committee for accepting issue 4/12 are:

(a) Adoption of the total ACI 318 Standard is appropriate because seismic resistance is considered in the overall development of the 318 Standard, including Appendix A on special provisions for earthquake resistance.

(b) Existing ATC 3 Chapter 11 originated from an early draft of a proposal by an ACI 318 Seismic Subcommittee to update the ACI 318 seismic design provisions. The basis of existing Chapter 11 was work developed under the guidance of Dr. Mete Sozen who served on the original ATC Concrete Task Group. Dr. Sozen is current Chairman of the ACI 318 Seismic Subcommittee which has the prime responsibility for the new proposed Appendix A of ACI 318. The ACI 318 Seismic Subcommittee worked towards producing a document that would be acceptable to the two professional communities involved--ACI and SEAOC. Two members of SEAOC, Clarkson Pinkham and Loring Wyllie serve on the 318 Seismic Subcommittee to provide SEAOC and ATC technical perspectives to ACI 318.

(c) The ACI 318 Standard is prepared and continuously updated in accordance with a rigorous consensus procedure approved by the American National Standards Institute and designated as ANSI/ACI 318-77 (A89.1). The ACI 318 Standard is unique among material design specifications in this regard. Because of the extensive review and adoption procedure, ACI 318 represents the state-of-knowledge for reinforced concrete and is widely adopted by model building code groups to regulate concrete design and construction.

(d) Membership of the ACI Building Code Committee has a wide geographical representation, with input from design professionals (including prominent engineers from earthquake-prone areas), educators, researchers, material and construction industries, government agencies, and building officials. The consensus procedure under which the document is prepared draws from the best documented data available.

5. The SEAOC representative on Committee 4, Mr. Wyllie, opposes issue 4/12 because he questions the wisdom of adopting for trial design a new set of provisions (Appendix A) that are incomplete in certain details, have not been thoroughly reviewed and adopted by Committee 318 of ACI, and are unaccompanied by detailed technical justification. He also believes the proposed change is a technical weakening of the existing Chapter 11 provisions.

Mr. Wyllie did, however, join with the majority of Committee 4 in unanimously adopting the following resolution: "Regardless of subsequent actions, it is the firm intent of Committee 4 that the final version of ACI 318 Appendix A, with appropriate modifications, be incorporated into ATC 3-06 aftr trial design."

6. The ATC representative, Dr. Bertero, did an outstanding job of working with the Committee and assisting it in developing desirable revisions to the existing Chapter 11. However, from the beginning he adamantly opposed substituting the proposed revision to ACI 318 Appendix A for the existing Chapter 11 contending that such an action was not within the scope of the Committee's mission. Dr. Bertero did not attend the last meeting of the Committee at which issue 4/12 was finalized and letter balloted. Instead

he sent in a letter of resignation from the Committee, accompanied by six pages of detailed comments on the proposed Chapter 11, the changes necessary to integrate it into the remainder of ATC 3-06, and the changes desirable in Appendix A. He concluded that:

> THE UPDATED DRAFT OF CHAPTER 11 SUBMITTED BY FINTEL ON MAY 29, 1980 AS SUGGESTED BY THE INDUSTRIES, CANNOT BE ACCEPTED FOR INCORPORATION TOGETHER WITH A NEW APPENDIX A INTO THE ATC 3-06.
>
> Even if new drafts of this chapter and Appendix A, including all the corrections, additions and clarifications suggested in the attached comments, are prepared, it would be a mistake to introduce them into ATC 3-06 for the TRIAL DESIGN PHASE. The main reason is that the designers will have to consider two new and very confusing cross references (Chapter 11 and the new Appendix A) which would increase the probability of misinterpreting the provisions. Even if designers are able to interpret correctly the interfacing provisions of the new Chapter 11 and Appendix A, no significant technical improvement in the design will be obtained.

Dr. Bertero did, however, agree that Chapter 11 of ATC 3-06 needed to be updated and integrated with the new Appendix A. He recommended that:

> A TECHNICAL SUBCOMMITTEE, WITH MEMBERS FROM COMMITTEE 4 AND THE ACI COMMITTEE THAT HAS PREPARED THE NEW APPENDIX A, BE FORMED AND CHARGED WITH THE MISSION OF IMPROVING AND INTEGRATING THE NEW APPENDIX A INTO CHAPTER 11 OF ATC 3-06.

7. At their final meeting, the majority of Committee 4 disagreed with Dr. Bertero's conclusion. They feel that significant technical improvements in design will be obtained by using the recommended Chapter 11, including Appendix A, as compared to the existing Chapter 11. Examples of technical improvements in Appendix A are the simpler rules for anchorage of bars, more realistic rules for the design of joints of frames and recognition that the strong column-weak beam frame is not always practical and a strong beam-weak column frame is sometimes necessary. In addition, in the sections of the recommended Chapter 11 providing transition from the existing ATC 3-06 to Appendix A there are important new concepts, approved by Dr. Bertero and Mr. Wyllie, and covering:

> (1) the use of flat plate framing systems for buildings assigned to category B,
>
> (2) precast and/or prestressed concrete framing systems for categories B, C and D, and
>
> (3) the use of precast floors as structural diaphragms.

Failure to utilize the best available state-of-knowledge for trial designs would not be in the national interest. Given the nature of the differences between the existing Chapter 11 and the proposed Chapter 11, incorporating Appendix A, the costs of designs that provide adequate seismic resistance for large portions of the country would be considerably greater for the existing than proposed Chapter 11. Action approving ballot issue 4/12 supersedes actions on issues 4/13 through 4/19.

July 18, 1980

Comment:

From: Robert N. Sockwell, AIA
Chairperson: Committee 8, Architectural, Mechanical and Electrical

Re: Clarification of Ballot Item 8/6
Section: 8.4, Elevator Design Requirements
Sub. Sec: 8.4.1, Reference Documents

This is a response to questions raised by members of the Joint Committee concerning the appropriateness of the wording of 8.4.1. Although it may appear that ANSI A17.1 (ANSI Safety Code for Elevators) is being selected as the singular acceptable design standard for elevators, that was not our intent. The committee considered A17.1 to be the most up-to-date and appropriate standard to reference for the seismic design of elevator systems. We intend to reword 8.4.1 to clarify its intent before ATC-3 becomes a final document.

I feel that the present wording of 8/6 will not impact the trial design phase, and ask that you vote affirmative on this issue.

[redacted]

7/16/80

[redacted]

ATC POSITION REGARDING CHANGES TO CHAPTER 11
OF ATC-3-06 AS PROPOSED BY TECHNICAL
COMMITTEE 4, CONCRETE

Technical Committee 4 proposes that a draft of Appendix A, ACI Standard 318-77 be substituted for Chapter 11 of ATC-3-06. ATC strongly opposes the proposed change for the following reasons:

1. The objective of the NBS Work Plan for Review and Refinement of Tentative Seismic Provisions is to review and refine the ATC-3-06 provisions so that they will be suitable for conduct of comparative trial designs. As stated in the Work Plan:

   "It is important to realize that this activity is neither the development of a draft standard nor the affirmation of a standard, only a mid-course adjustment thought necessary for wise expenditure of research funds for the conduct of trial designs."

   The purpose of the trial designs is to help make an assessment of new concepts and new requirements embodied in the Tentative Provisions. The proposed substitution of a draft document for Chapter 11 does not fall within the objectives or scope of the NBS Work Plan. The draft Appendix A does not introduce any significant technical improvement as far as seismic design of concrete buildings is concerned.

2. In carrying out the review and refinement it has been agreed that changes to the provisions should not be made just for the sake of changing or because a particular provision(s) is not liked. Rather changes are to be made to clarify the intent of a provision or to make a technical correction. There is no technical justification for making the proposed substitution and it does not clarify the intent.

3. Some of the draft Appendix A provisions are less restrictive than those in Chapter 11. No technical justification has been presented for reducing the Chapter 11 requirements.

ACI Committee 318 and

4. The proposed Appendix A substitution is still a draft because it has not been accepted by the ACI membership. Therefore it is not a "consensus" document. Therefore, if the proposed substitution is made, the trial designs would be conducted using provisions that are still to be finalized.

5. The proposed substitution is premature. After ACI 318-77, Appendix A is adopted by the ACI membership and thus becomes a "consensus" document, the question of substituting its provisions for those in Chapter 11, ATC-3-06 can be addressed.

6. The comment has been made that ATC-3-06 Chapter 11 is merely an outdated draft of Appendix A, because Chapter started from an early draft of Appendix A. This comment reflects a misunderstanding of Chapter 11 – possibly because Committee 4 members have not carefully compared the two documents. On the other hand, the draft Appendix A is not an upgrade of the ATC-3-06 Chapter 11 because there are a number of less restrictive design provisions in Appendix A.

7. The flat plate and precast-prestressed provisions should not be added to the ATC-3-06 Chapter 11 until they have been reviewed by the appropriate professional committees including ACI.

8. There has not been sufficient time to review the proposed May 28 substitution. As noted during the July 16 meeting at NBS (Joint Committee) the proposed substitution is not a "seasoned" document; it needs exposure to careful professional committee study and review. Chapter 11 of ATC-3-06 resulted from three years of study and review.

R. L. SHARPE, ATC.

Notes of presentation to ATC-3-06 Review NBS July 16, 1980

Committee No. 1- Seismic Risk

I appreciate the opportunity of addressing the assembled members of the different project committees and all of the parties interested in seismic design improvements.

In responding to the committee decisions and the results of their vote it should be pointed out that the committee only met once. This was at the organizational session here at NBS last December. It may be presumptuous but if we had met I believe I could have at least tempered some of the decisions. The ATC-3 document was the result of many compromise decisions and if neither the need for compromise nor the results of this compromise are recommended by all committees then the current NBS effort will ultimately be non-productive. The primary aim of the review of the ATC-3-06 provisions is to enable trial designs to proceed more efficiently. As the actions of Committee 1 are primarily related to seismic zoning they do not have any influence of the trial designs. However, the committee has raised several issues and voted on them so as a major ATC-3 participant they should be responded to.

The major thrust of actions presented to the committee for consideration were suggestions, possibly by those with vested interests, for lowering the criteria. The only unanimity appears to be for less. The committee voted not to approve the suggested change in the Seismic Performance Category Table, Table 1-A and voted against most of the changes suggested for Table 1-B except those relating to areas of lower seismicity (Map Areas 1 and 2). In this regard, a misconception was circulated that the seismicity in zone 1 was zero and that the effective peak acceleration values in Map Area 2 are actually less than 0.05g. The contours are given in Appendix C1-3 and all areas in Map 2 lie within a contour value of 0.05g or greater. The misconception regarding areas of zero seismicity was one that our ATC-3 Task Group battled with even to the point of unsuccessfully recommending that the number sequence of the map areas be reversed so that zero was not even available. From purely seismological terms there is no state among the 50 in the Union in which earthquakes have not been recorded. Low seismicity we can agree with but not zero seismicity.

Two additional changes apart from alterations to the Tables on page 35 of ATC-3-06 were made and voted upon. The first of these was to base the zoning map directly on contours rather than the present county-by-county procedure. This same recommendation was made to the full ATC-3 group when the maps were first developed but despite diligent effort they were not accepted. The primary reasons are procedural. Contours would have to be extended outside the United States and closed to allow interpolation to be performed in border areas. Some forms of construction are only permitted below certain design levels. These zonations are not distinct when contours are used. The ATC-3-06 report recognized the difficulties with a county-by-county basis and on page 29 of the recommended procedures gave an alternate version which does not require any direct application of a map. Human nature being what it is, and as has been demonstrated by this present review, effort will be spent lobbying for lower, not higher numbers so if ATC-3 were adopted the alternate procedure could be invoked to reduce discrepancies.

The second request was to include in the guidelines a definition of an active fault. I am inalterably apposed to this suggestion. The question of what constitutes an active fault is not one upon which there is broad professional agreement so it should not be "defined" within a code. The suggested definition is inadequate for another fundamental reason. Consideration of the frequency of movement and the amount of movement are not considered. A fault which may move by an inch or two is certainly not as hazardous as one which may move several feet.

Thank you for your attention.

NEVILLE DONOVAN, ATC

Ballot item 4/3: Committee 2 considered a similar proposal to change the vertical load requirement for horizontal prestressed members. The minutes of that discussion follow:

Section 3.7.12, Vertical Seismic Motions: The proposed modification submitted by Professor Hawkins through committee 4 was considered. Harris questioned whether the proposed really stated what Hawkins really meant. Holmes speculated that the intent was to apply an upward seismic force of $0.2Q_D$, not an upward net force of $0.2Q_D$. Iyengar pointed out that the stated reason did not correspond to the recommended change. The proposal was tabled and Fintel was charged with contacing Hawkins prior to the Thursday session. Fintel was unable to contact Hawkins, and the proposal received no further consideration.

Ballot item 4/6: Committee 3 considered a very similar proposal, i.e. to make the Category D requirements for piles the same as for Category C. Their action:

11d Discussion

The committee considered the comments that were received, but decided that additional conservatism was required for Category D structures. Therefore, Section 7.6.1 should not be modified.

11e Recommendation

The committee recommended not to change or delete Section 7.6.1

Ballot items 4/8 and 4/9: Committee 8 considered the same proposals. This sheet summarizes their action. First, the original text of the proposals:

<u>SEC. 8.2.3</u> Add the following sentence at the end of this section: "Connector fasteners shall develop elastic forces resulting from twice the loads determined from Section 8.2.2 above."

BASIS: Current practice as outlined in UBC-79.

<u>SEC. 8.2.2</u> Add the following sentence at the end of this section: "The force Fp shall be applied in the vertical direction, as well as longitudinally and laterally, in combination with the static load of the element."

BASIS: UBC-79; The effect of vertical acceleration should be included in design of non-structural components and systems.

Excerpt from the response to the proposal by the ATC representative for Committee 8:

- <u>Section 8.2.2</u> FORCES

  I disagree with this comment. The F_p, on parts of structures, is normally considered as a horizontal force, from any direction.

- <u>Section 8.2.3</u> EXTERIOR WALL PANEL ATTACHMENT

  I disagree with this comment. C_c value for wall attachment is already high and beyond this, detailing ductility is the important requirement.

'ction taken by Committee 8:

Issue 1 Section 8.2.3 <u>Exterior Wall Panel Attachment</u>

A motion was made by Swatta, seconded by Wintz that the ATC recommendation for no changes be accepted. Motion passed 5-0.

Issue 2 Section 8.2.2 <u>Forces</u>

Motion made by Swatta and seconded by Wintz, passed 5-0, to accept ATC recommendation for no changes. The argument was nonpersuasive as no specific reference was given for UBC-79 practice.

BALLOT ITEMS S/1 through S/5: Committee 2 considered each of these proposals. The following shows the results of their ballot and is excerpted from the minutes of their third meeting. Item 2 below corresponds to ballot item S/1 , item 3 corresponds to ballot item S/2, etc.:

Ballot Item	Initial Tally*		Final Tally*		Remarks Following
	yes	no	yes	no	
1. S. Perf. Category	5	3	0	8	a
2. S. Index, Area 5	3	5	2	6	a
3. S. Index, Area 4	2	6	2	6	a
4. S. Index, Area 3	2	6	2	6	a
5. S. Index, Area 2	4	4	2	6	a
6. S. Index, Area 1	2	5	0	7	a

Notes on the ballot items:

a. Harris pointed out that further consideration of the discussion in Phoenix convinced him that item 1 should have been contingent on item 6 alone. The committee agreed. None of the changes proposed for the seismicity index carried on the initial tally, and the sentiment of the committee shifted somewhat towards recommending no change for the purpose of trial design. Iyengar felt that item 5, changing the seismicity index for map area 2 from a 2 to a 1 was quite important; he felt that many locations in that map area would unilaterally decide to ignore seismic provisions such as those included for seismicity index 2.

UNITED STATES DEPARTMENT OF COMMERCE
National Bureau of Standards
Washington, D.C. 20234

July 24, 1980

MEMORANDUM TO: Participants in the Review and Refinement of the Tentative Seismic Provisions

Dear Participant:

This package allows you to complete balloting the proposed refinements to the ATC-3-06 provisions. Recall that Committee 5 (Masonry) has reformated their ballot items and documented each proposed change so that your task of completing the ballot will be easier. Also recall that Committee 4 (Concrete) has proposed seven supplementary ballot items for proposed changes in Chapter 11 of ATC-3-06. These supplementary items each passed Committee 4 but were not included in the joint ballot because they were supplanted by joint ballot item 4/12 (the direct reference to Appendix A of ACI 318). You are requested to ballot on these supplementary items with the understanding that they will be counted only if item 4/12 does not pass.

Attached to this letter are:

- A summary of the actions of Committee 5 (Masonry), written by the secretary of the committee.

- A general comment from ATC on the Committee 5 ballot.

- A comment from ATC on the supplementary items from Committee 4 (Concrete).

Enclosed in this package are:

- Two ballot tally sheets, one to be returned to NBS in the attached return envelope.

- The supplementary ballot items from Committee 4.

- The version of Chapter 12 marked with Committee 5's ballot items 5/1 through 5/28.

- A tabulation of the Committee 5 comments and the ATC comments on the ballot items in Chapter 12.

- The version of Chapter 12A marked with Committee 5's ballot items 5A/1 through 5A/81.

- A tabulation of the Committee 5 comments and the ATC comments on the ballot items in Chapter 12A.

Please complete and return this ballot as soon as possible. It must be postmarked no later than August 8, 1980, to be counted. Once again, comments are welcome. If you have any questions, please call Dr. J. R. Harris at (301) 921-2170.

Sincerely,

E O Pfrang

Edward O. Pfrang, Chairman
Joint Committee on Review and Refinement

Enclosures

Record of Ballot History of Committee 5

The following is an abbreviated record of how Committee 5 arrived at the enclosed ballot document.

- At the initial December 11 meeting of Committee 5, two points were agreed upon by the Committee. These are:

  1. It is invalid to use ultimate strength design for masonry.

  2. Chapter 12A is not an acceptable document for masonry design in nonseismic areas.

- On January 4 a task group met in Northbrook, IL to further study the points described in the December 11 meeting of Committee 5. The task group made the major decision that they would rework Chapters 12 and 12A even though that meant using the ultimate strength concept. The task group had reservations, but they did agree to move in that direction. At the task group meeting, it was still agreed that Chapter 12A, as it existed, was not a workable document. The task group, however, was divided into two equal groups that each wanted to go their own direction. These two groups agreed to prepare proposals for the next committee meeting.

- At the February 21 and 22 meeting, two presentations were made to the committee with regard to Chapter 12A. One proposal involved modifying ACI 531 on masonry design so that it would include both clay and concrete products and substituting that for Chapter 12A. The second proposal was presented by a group that had carefully examined Chapter 12A and concluded that it was possible to modify it for use in trial designs. That group proved its point by presenting a detailed examination of Chapter 12A and the needed revisions. After much discussion, Committee 5 agreed unanimously to proceed with the proposal to modify Chapter 12A.

- The committee next met in Denver on March 21 and 22. At this meeting Chapter 12A was finished and Chapter 12 was examined in some detail and prepared for ballot. At the conclusion of this meeting, both Chapters 12 and 12A were in the committee's hands for ballot.

  It should be noted at this point that the committee discussed all items in Chapters 12 and 12A at the Arlington, TX and Denver, CO meetings to determine changes that should be balloted in written form. Thus, prior to written ballot, each item was voted on in a committee meeting. The next stage was to collect written ballots with written comments.

- The next meeting of the committee was held in Chicago on May 16. The committee held an 18 hour session in Chicago and completed discussion and reballoting of all items of Chapter 12A. At that point, the ATC representative asked that the committee set aside its prior ballots on Chapter 12 and give ATC the opportunity to prepare a new Chapter 12 which considered changes the committee had made in Chapter 12A. The committee agreed to do this and set the next meeting for Washington, DC to discuss the ATC proposals.

- The committee next met in Washington, DC on June 5 and 6. Prior to taking up Chapter 12, the committee concluded its discussions of Chapter 12A by incorporating design guidelines for hollow clay masonry. The committee then took up the ATC proposal for Chapter 12. The committee went through, item by item, all proposals for Chapter 12 that were presented by the ATC representatives to the committee.

- The enclosed ballot document indicates every item that was balloted in written form by the committee and the resulting vote count. At the July 16 and 17 meeting of the full Joint Committee, a ballot was presented and discussed in limited detail by the committee and by the ATC representative. At that meeting, it was made clear by the attendees that the ballot document that had been put together grouped too many individual changes into each ballot item. The full Joint Committee indicated that it would prefer to ballot on a larger set of items, each with a small number of changes, so that the work of the committee could be considered in more detail. Committee 5 then held an evening session on July 16 and a session on July 17 and reported back to the Joint Committee that they could indeed provide the material requested. The enclosed ballot document is the result of the efforts of Committee 5 and the ATC representatives to prepare the requested material.

APPLIED TECHNOLOGY COUNCIL'S COMMENTS

ON COMMITTEE 4 BALLOT ITEMS

ITEM 4/15: The new section 11.9 as worded would apply to all categories of buildings, A through D, which would make the new section applicable to high seismic risk areas. This is a sweeping change and is going too far too fast. For applications in high seismic risk areas, such as California, these provisions should be carefully reviewed and studied by professional committees, such as the SEAOC Seismology Committee. ATC therefore strongly recommends that the new section 11.9 not be adopted as written, but that it be modified to apply only to category A and B buildings until more study can be given. This recommendation has resulted from further consideration by the ATC representatives.

ITEM 4/19: The wording of the proposed addition to the commentary should be modified as follows to make it clearer: in the second line change "will" to "should," and in the third line, insert "are" before "greater." The reason for changing "will" to "should" is that the drift limits of Table 3-C would be changed if ballot item 2/22 is adopted. ATC concurs with the proposed addition to the commentary as modified above.

APPLIED TECHNOLOGY COUNCIL'S COMMENTS
ON COMMITTEE 5 BALLOT ITEMS

INTRODUCTION

Chapter 12A was included in the ATC-3-06 report because a nationally applicable design standard for masonry was and is still not available. Thus Chapters 12 and 12A were developed as complementary documents. Some seismic provisions were included in Chapter 12A and thus many of the bollot items involve a transfer of these items to appropriate Sections of Chapter 12.

Many of the other ballot items involve minor and editorial changes and ATC concurs with most of these. However, there are a number of other ballot items that ATC believes are substantive changes to the document. Comments on these items are in the accompanying tabulation of comments.

There are two important items that have not received adequate consideration by the committee at this time. These are the allowable stresses for unreinforced masonry and the design requirements for hollow unit masonry.

ATC therefore strongly recommends that Committee 5 continues its deliberations beyond the meeting of July 16th and 17th. This will ensure that consistency, cross-referencing and definitions have been adequately covered as a result of the large number of changes and that the two items referred to above can be adequately addressed.

APPENDIX C

NEW PROPOSALS RECEIVED AT THE FINAL MEETING OF THE JOINT COMMITTEE

Two proposals for change to the Tentative Provisions were received by the Joint Committee at their final meeting on July 16-17, 1980. They are:

1. "Proposed Revisions to Chapter 7 Foundations" submitted by Frank M. Fuller of Raymond International Builders.

2. "Recommendations to Incorporate Provisions for the Eccentric Braced Frame in ATC 3-06 Tentative Provisions for the Development of Seismic Design for Buildings, June 1978," submitted by William Smith of the American Iron and Steel Institute.

These two proposals are included in this appendix.

Because these proposals were not acted upon by any of the technical committees, the Joint Committee did not ballot them. They are included here for future consideration.

Proposed Revisions to Chapter 7 Foundations

ATC-3-06 Report

Submitted by

F. M. Fuller
Raymond International Builders Inc.

(1) Section 7.4.4 (A) UNCASED CONCRETE PILES: Revised to read as follows:

(A) UNCASED CONCRETE PILES: Reinforcing steel shall be provided for uncased cast-in-place concrete piles, drilled piers or caissons with a minimum of four No. 5 bars with a minimum steel ratio of 0.005 for pile diameters up to 24 inches and 0.0025 for diameters of 24 inches or greater. Such steel shall be enclosed in a steel spiral of minimum No. 2 bar and a maximum 6-inch pitch except the maximum pitch for the top two feet shall be 3 inches.

COMMENT: Seismic reinforcing should be full length for concrete piles not enclosed in a steel casing. If ground motions starts from below the pile tip, any unreinforced lower portion of the pile will readily crack and may be displaced resulting in possible foundation failure.

The current requirements for minimum steel ratio in the ATC Report were based on the assumption that uncased piles would be of relatively large diameter. Inasmuch as the eventual seismic regulations will apply nation wide, it must be recognized that uncased cast-in-place concrete piles are being installed with conventional diameters comparable to metal cased piles. Therefore, the minimum steel requirements have been increased for piles less than 24 inches in diameter.

The above revision permits only the use of a steel spiral for lateral steel and specifies a minimum bar diameter which is not in the current wording. Also, the maximum pitch has been decreased for both the body of the pile and the top two feet. The main purpose of the spiral steel under seismic loading is to confine the core concrete and provide a measure of ductility to the pile. This acts to some degree like the metal casing although it is not as efficient or effective.

The maximum 6 inch pitch is necessary to get a reasonable degree of confinment of the core concrete. The 6 inch spacing makes the use of a spiral much more practical than individual hoops. Square ties should not be permitted because they offer no confinment.

(2) Section 7.4.4 (B) METAL CASED CONCRETE PILES: In the last line change "3 inches" to "4 inches".

COMMENT: The steel casing provides full confinment and therefore the 3-inch spacing is unnecessary.

(3) Section 7.5.3 (A) UNCASED CONCRETE PILES: Revise to read as follows:

(A) UNCASED CONCRETE PILES. Reinforcing steel shall be provided for uncased cast-in-place concrete piles, drilled piers or caissons with a minimum of four No. 6 bars with a minimum steel ratio of 0.0075 for pile diameters up to 24 inches and 0.005 for diameters of 24 inches or greater. Such steel shall be enclosed in a steel spiral of minimum No. 3 bar for pile diameters up to 24 inches and No. 4 bar for piles of 24-inch diameter or greater. Spiral steel shall have a maximum pitch of 6 inches except that the maximum pitch for the top four feet shall be 3 inches.

COMMENT: The same basic comments as stated above under Item 1 apply to this Item. The increase in steel ratio for piles less than 24 inches in diameter recognizes the use of small diameter uncased concrete piles.

(4) Section 7.5.3 (B) METAL CASED CONCRETE PILES: In the last line change "3 inches" to "4 inches".

COMMENT: See comment for Item 2 above.

AMERICAN IRON AND STEEL INSTITUTE
1000 16TH STREET, N. W.
WASHINGTON, D. C. 20036

RECOMMENDATIONS TO INCORPORATE PROVISIONS FOR THE ECCENTRIC BRACED FRAME IN ATC-3-06 TENTATIVE PROVISIONS FOR THE DEVELOPMENT OF SEISMIC DESIGN FOR BUILDINGS, JUNE 1978

The very worthy, steel eccentric frame does not appear to be classifiable anywhere under the systems described on page 52. This is unfortunate since ATC-3-06 is purported to represent the state-of-the-art. It should also be recognized that extensive analytical studies coupled with cyclic testing of large-sized subassemblages was thoroughly reported on by the University of California as far back as 1977. Several papers have since been published. The system is presently being used in construction of hospitals, banks, hotels and office buildings in areas of high seismicity.

The following recommendations are those presently being proposed for incorporation in the Recommended Lateral Force Requirements of the Structural Engineers Association of California and the City of San Francisco Building Code now undergoing updating.

Table 3-B. Response Modification Coefficients. To accommodate the eccentric braced frame, the following additional entries are recommended for Table 3-B:

TABLE 3-B

RESPONSE MODIFICATION COEFFICIENTS[1]

Type of Structural System	Vertical Seismic Resisting System	Coefficients R^7	$C_d{}^8$
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with shear panels	6½	4
	Shear walls		
	Reinforced concrete	4½	4
	Reinforced masonry	3½	3
	Braced frames	4	3½
	Unreinforced and partially reinforced masonry shear walls[6]	1¼	1¼
BUILDING FRAME SYSTEM: A structural system with anessentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with shear panels	7	4½
	Shear walls		
	Reinforced concrete	5½	5
	Reinforced masonry	4½	4
	Braced frames	5	4½

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[7]	C_d[8]
BUILDING FRAME SYSTEM (con't)	Eccentric braced frames designed in accordance with the provisions in Section 10.7.	6	5
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special moment Frames		
	Steel[3]	8	5½
	Reinforced concrete[4]	7	6
	Ordinary moment frames		
	Steel[2]	4½	4
	Reinforced concrete[5]	2	2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Shear walls		
	Reinforced concrete	8	6½
	Reinforced masonry	6½	5½
	Wood sheathed shear panels	8	5
	Braced frames	6	5
	Eccentric braced frames designed in accordance with the provisions specified in Section 10.7	8	5½
INVERTED PENDULUM STRUCTURES. Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Special Moment Frames		
	Structural steel[3]	2½	2½
	Reinforced concrete[4]	2½	2½
	Ordinary Moment Frames		
	Structural steel[2]	1¼	1¼

[1]These values are based on best judgement and data available at time of writing and need to be reviewed periodically.
[2]As defined in Sec. 10.4.1.
[3]As defined in Sec. 10.6.
[4]As defined in Sec. 11.7.
[5]As defined in Sec. 11.4.1.
[6]Unreinforced masonry is not permitted for portions of buiodings assigned to Category B. Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Chapter 12.
[7]Coefficient for use in Formula 4-2, 4-3, and 5-3.
[8]Coefficient for use in Formula 4-9.

Add the following change to Section 3.3.4(A), which sets forth the types of framing systems that may be used:

3.3.4 SEISMIC PERFORMANCE CATEGORY C

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

(A) SEISMIC RESISTING SYSTEMS. Seismic resisting systems in buildings over 160 feet in height shall be one of the following:

1. Moment resisting frame system with Special Moment Frames.

2. A Dual System.

3. A system with structural steel or cast-in-place concrete braced frames or shear walls in which there are braced frames or walls so arranged that braced frames or walls in any plane resist no more than 33 percent of the seismic design force including torsional effects; this system is limited to buildings not over 240 feet in height.

4. A building frame system in which the seismic design lateral force including torsional effects is resisted by eccentric braced frames.

Add a new Section 10.7 to provide design criteria for the eccentric braced frame referenced in Table 3-B.

10.7 STEEL ECCENTRIC BRACED FRAMES

Eccentric braced frames shall be designed in accordance with all the provisions of Section 10.6, in addition to the requirements which follow:

1. Definitions.

LINK BEAM is that part of the beam which is designed to yield in shear.

2. Design Criteria.

a) The link beam should be designed to yield in shear at a force not less than that obtained from an elastic analysis of the frame, using the laterial forces as prescribed by these provisions. The shear yield shall occur prior to bending yield. The link beam web should be a single thickness, without doublers.

b) Eccentric Braced Frames utilizing bending yielding of the link beam may utilize the "R" values as described in Table 3-B if energy absorption and ductility equivalent to that obtained from the shear yielding systems can be demonstrated. Otherwise they shall be assigned "R" values the same as concentric braced frames.

c) Brace members shall be designed for at least 1.5 times the axial forces and moments which occur at onset of link beam yielding.

d) Columns shall be designed using the "strong column - weak beam" concept, with the beam plastic moment capacity limited to a reduced M_p consistent with that allowed by the yielded web.

e) All of the requirements of Part 2 of Ref. 10.1 (Plastic Design) shall be adherred to. Particular attention should be given to lateral bracing of the beams and to the d/t ratio of the link beam web. The need for web stiffeners in the link beam web shall be considered. A lateral brace must be placed at the point of intersection of the eccentric brace with the beam unless adequate bracing by some other system can be properly substantiated.

f) Axial forces, when occurring in the link beam, shall be considered in the design.

g) Appropriate consideration should be given to the collapse mechanism for extreme loading conditions.

h) Beam to column connections shall develop the full plastic moment capacity of the beam and the full shear yield capacity of the web. Column flange and web stiffeners should be designed according to the usual methods.

i) Brace to beam connections will depend on the brace section chosen. Primary requirements are that a pair of stiffeners be provided in the beam at the side of the connection defining the end of the link beam and that brace forces be transferred directly to these stiffeners by a flange (either a flange of the brace or the flange of a tee-shaped gusset). In no case should any component of the connection extend into the link beam zone. The connection should be designed with the additional factor of safety required for the brace.

The foregoing recommended additions to ATC-3-06 are based on the recommendations now being considered by the Seismology Committee of the Structural Engineers Association of California. Every effort has been made to make these additions conform in all respects to those recommendations.

Committee on Construction Codes and Standards
AMERICAN IRON AND STEEL INSTITUTE
July 11, 1980

REFERENCES ON THE ECCENTRIC BRACED FRAME

C. W. Roeder and E. P. Popov. "Inelastic Behavior of Eccentrically braced Steel Frames Under Cyclic Loading," EERC Report 77-18, University of California, Berkeley, August 1977.

C. W. Roeder. "Inelastic Behavior of Eccentrically Braced Frames Under Cyclic Loadings," Ph.D. Thesis, University of California, Berkeley, 1977.

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C. W. Roeder and E. P. Popov. "Cyclic Shear Yielding of Wide Flange Beams," Journal of Engineering Mechanics, ASCE, Vol. 104, No. EM4, August 1978.

Edward J. Teal. Practical Design of Eccentric Braced Frames to Resist Seismic Forces. Structural Steel Educational Council, California Field Ironworkers Administrative Trust, 1980.

STRUCTURAL ENGINEERS ASSOCIATION
OF NORTHERN CALIFORNIA

2 June 1980

To: Fritz Matthieson, Chairman Seismology Committee

From: Mark Saunders, Chairman Eccentric Braced Frame Subcommittee

ECCENTRIC BRACED FRAME SUBCOMMITTEE

FINAL REPORT

HISTORY

This Subcommittee was formed as an ad hoc committee under the Ductile Frame Subcommittee in March 1979. The original and primary charge of the committee was to develop a "K" factor for eccentric braced frames (EBF's). The Ad Hoc Committee met in March and May of 1979 and in June drafted preliminary recommendations which were presented briefly to the State Seismology Committee. For the 1979-80 year, the Ad Hoc Committee became a full subcommittee of the Seismology Committee. We have met three times during the year (November 1979, and February and April 1980), have completed our recommendations for "K" factors, and have drafted recommended code modifications and additions.

APPROACH

According to the Blue Book, the purpose of the "K" factor "... is to give all types of structural systems an equal probability of performance under a designated earthquake." Thus, it is a relative factor and our approach was to place the EBF in its appropriate relative position in the "K" factor table. In order to do this, we mainly considered ductility; however, considerations of redundancy and the effect of the "K" factor on overturning and diaphragms were also incorporated.

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EXECUTIVE SECRETARY – BILL GILES

RECOMMENDATIONS

Attached hereto is a copy of the "K" factor table (TABLE 1-A), revised per our recommendations. Also attached is a draft of recommended code language to be incorporated in the Blue Book. After this material has been reviewed by all of the appropriate bodies, we will undertake to write the final code language and the commentary.

ECCENTRIC BRACED SUBCOMMITTEE

Messrs. Alsmeyer, De Maria, Epstein, Kodavatiganti, Krawinkler, Moore, Nicoletti, Nordenson, Popov, Shields, Smith, Warren, Weathers, Merovich, Doig, Greenwood, Chang.

Ms. Kozler.

MS/hc

Attachments: a/s

ECCENTRIC BRACED FRAME SUBCOMMITTEE
SEAONC

RECOMMENDED "K" FACTOR TABLE
INCORPORATING ECCENTRIC BRACED FRAMES

DRAFT

TABLE 1-A

HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES

TYPE OR ARRANGEMENTS OF RESISTING ELEMENTS	Value of "K"
Building with a box system as defined in Section 1(B).	1.33
All building framing systems not otherwise classified herein.	1.00
Buildings with eccentric braced frames designed in accordance with the following criteria: the eccentric braced frames shall have the capacity to resist the total required lateral force and shall be designed in accordance with the provisions specified in this code.	0.80
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames designed in accordance with the following criteria:	
1. The frame and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames.	

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	Value of "K"
2. The shear walls or braced frames acting independently of the ductile moment resisting space frame shall resist the total required lateral force.	
3. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.	0.80
Buildings with a dual bracing system as described above, except utilizing eccentric braced frames in lieu of shear walls or conventional braced frames. The eccentric braced frames shall be designed in accordance with the provisions specified in this code.	0.67
Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: the ductile moment resisting space frame shall have the capacity to resist the total required lateral force.	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building. (1) (2)	2.50
Structures other than buildings and other than those set forth in Table 1-B.	2.00

RECOMMENDED ADDITIONS AND MODIFICATIONS
TO THE BLUE BOOK

DRAFT

Section 1(B) Add a definition as follows:

"Eccentric Braced Frame is a braced frame complying with the requirements given in Section 5 of these Recommendations."

Section 1(J)1. Revise to read as follows:

"a. Force Factor. All buildings designed with a horizontal force factor K = 0.67 or 0.80, shall have ductile moment resisting space frames or eccentric braced frames."

After paragraph "g", insert the following:

"Eccentric Braced Frames. Design of eccentric braced frames shall conform to the requirements of Section 5 of these Recommendations. Steel used shall be as required under braced frames above.

Section 5
(DRAFT)

STEEL ECCENTRIC BRACED FRAMES

(A) General

All of the provisions of Section 4 shall apply to eccentric braced frames in addition to the requirements which follow.

(B) Definitions

LINK BEAM is that part of the beam which is designed to yield in shear.

(C) Design Criteria

1) The link beam should be designed to yield in shear at a force not less than that obtained from an elastic analysis of the frame, using the lateral forces as prescribed by this

Section 5
DRAFT (cont.)

code, and factored to the plastic design level. The shear yield shall occur prior to bending yield. The link beam web should be a single thickness, without doublers.

2) Eccentric Braced Frames utilizing bending yielding of the link beam may utilize the "K" values as described above if energy absorption and ductility equivalent to that obtained from the shear yielding systems can be demonstrated. Otherwise they shall be assigned "K" factors the same as concentric braced frames.

3) Brace members shall be designed for at least 1.5 times the axial forces and moments which occur at onset of link beam yielding.

4) Columns shall be designed using the "strong column - weak beam" concept, with the beam plastic moment capacity limited to a reduced M_p consistent with that allowed by the yielded web.

5) All of the requirements of Part 2 of the AISC specification (Plastic Design) shall be adhered to. Particular attention should be given to lateral bracing of the beams and to the d/t ratio of the link beam web. A lateral brace must be placed at the point of intersection of the eccentric brace with the beam unless adequate bracing by some other system can be properly substantiated.

6) Axial forces, when occurring in the link beam, shall be considered in the design.

7) Appropriate consideration should be given to the collapse mechanism for extreme loading conditions.

Section 5
DRAFT(cont.)

8) Beam to Column connections shall develop the full plastic moment capacity of the beam and the full shear yield capacity of the web. Column flange and web stiffeners should be designed according to the usual methods.

9) Brace to beam connections will depend on the brace section chosen. Primary requirements are that a pair of stiffeners be provided in the beam at the side of the connection defining the end of the link beam, and that brace forces be transferred directly to these stiffeners by a flange (either a flange of the brace or the flange of a tee-shaped gusset). In no case should any component of the connection extend into the link beam zone. The connection should be designed with the additional factor of safety required for the brace.

Added References

1. Roeder, C.W. and E.P. Popov, Inelastic Behavior of Eccentrically Braced Steel Frames Under Cyclic Loadings, EERC Report 77-18, University of California, Berkeley, August 1977.

2. Roeder, C.W. and E.P. Popov, Eccentrically Braced Steel Frames for Earthquakes, Journal of Structural Division, ASCE, Volume 104, No. ST3, March 1978.

3. Roeder, C.W. and E.P. Popov, Cyclic Shear Yielding of Wide Flange Beams, Journal of Engineering Mechanics, ASCE, Volume 104, No. EM4, August 1978.

APPENDIX D

ORIGINAL WORKPLAN
FOR THE PROJECT

WORK PLAN FOR

REVIEW AND REFINEMENT OF TENTATIVE SEISMIC PROVISIONS

The Tentative Provisions for the Development of Seismic Regulations were developed by the Applied Technology Council (ATC) in an effort that included a wide range of experts in the actual drafting of the provisions. Two external review drafts were circulated to a large portion of the interested and informed community of eventual users. However, because the Tentative Provisions are quite innovative, serious doubts about them may exist. Prior to undertaking an expensive assessment of the Tentative Provisions through the conduct of trial designs, an attempt will be made to investigate these doubts and to improve the Tentative Provisions where possible. It is likely that many issues exist that will not be resolved by this activity; some will require the information that will be developed in the conduct of the trial designs and subsequent impact assessment before resolution can be expected. However, the activity can and should improve the likelihood that the trial designs and impact assessment will not be conducted with provisions containing flaws so serious that the results of the trial designs would be compromised. The activity can also serve to focus the trial designs on particularly controversial issues.

GENERAL DESCRIPTION

The proposed review and refinement is designed to be accomplished in less than a year, so that the overall assessment may proceed in a timely fashion. The committee structure for carrying out the assessment is shown in figure 1. Nine Technical Committees will be formed with interests that collectively cover the Tentative Provisions. The Joint Committee on Review and Refinement will consist of all voting members of the Technical Committees. The chairmen of the Technical Committees will form a Coordinating Committee.

The membership of each Technical Committee will be made up of representatives of organizations that have particular interest in the Tentative Provisions; the invited participants are listed on attachment 1. Because the individual Technical Committees are each formed of specialists and are focused on a small portion of the Tentative Provisions, they may not be well balanced. The full Joint Committee membership is more balanced, however, and the final and decisive ballot on any refinements will involve the entire Joint Committee. It is important to realize that this activity is neither the development of a draft standard nor the affirmation of a standard, only a mid-course adjustment thought necessary for wise expenditure of research funds for the conduct of trial designs and impact assessment.

In addition to the voting members, each Technical Committee will include a non-voting member from each of the following organizations: ATC, the Building Seismic Safety Council (BSSC) and the National Bureau of Standards (NBS). The ATC representative will be a technical resource to the committee. He will have been closely involved with the development of the provisions of interest to the committee. The NBS representative will be the technical secretary for each committee and will convene the first meeting and provide technical and administrative support throughout the effort. The BSSC representative will provide overview with appropriate committees of the BSSC. Each Technical Committee will elect a chairman from among its voting members and will develop its own agenda for review and refinement, within the general guidelines established for scheduling, for coordination of refinements, and for documentation.

The Coordinating Committee, will be formed from the chairmen of the nine committees and will be chaired by NBS. A representative of both ATC and BSSC will also be included, but neither of them nor the NBS chairman will be voting members. The Coordinating Committee will review all refinements proposed by the other committees with the express purpose of preventing the development of conflicts and inconsistencies. Their operation is described in more detail in subsequent paragraphs.

The Joint Committee will be headed by a nonvoting NBS chairman. NBS will also provide support for convening and operating the Joint Committee.

OBJECTIVES AND GUIDELINES

The Tentative Provisions employ several new concepts that need thorough assessment, which is what the trial designs are intended to help provide. The objective of this activity is to assure that the provisions used in the trial designs do allow such an assessment and to focus the trial designs on particularly controversial issues. It is intended that this activity assure, to the extent possible, that the information necessary to reach a consensus on the new concepts will be available following the trial designs. It is not expected that a consensus will be reached on all issues before the trial designs. Questioning the technical merit of a provision and questioning how effective a provision is in implementing a new concept are well within the scope of the activity. However, it is beyond the scope to radically alter the new concepts embodied in the Tentative Provisions.

The fact that the Technical Committees are established to refine the Tentative Provisions but are not encouraged to grossly change them requires the establishment of clear guidelines, which is difficult. The introduction to the Tentative Provisions includes lists of objectives and new concepts, which are on attachment 2. The best available guidelines seem to be that these objectives shall not be compromised nor shall sweeping changes be made to the new concepts. Refinements violating these guidelines will be discouraged. If such refinements do result they will be documented and the advice of BSSC will be sought concerning their use in the trial designs and impact assessment.

The purpose of this NBS activity is to review the ATC3-06 provisions and present a recommended set of seismic design provisions and test procedures to BSSC. NBS will provide to BSSC documentation on all actions made to revise the ATC documents. This includes both successful and unsuccessful proposed changes. Upon receipt of these recommendations the BSSC membership shall consider by ballot these recommendations and any revisions proposed to them according to BSSC adopted procedures. The overview committee, its members or delegates shall be afforded the opportunity to participate in any and all activities with timely access to all documents and correspondence. The overview committee will periodically advise NBS of its observation on the conduct of the activity and as required recommend to the BSSC Board corrective actions. Board recommendations to NBS will be resolved to both parties satisfaction. No trial designs will be conducted under this overall program until the BSSC has approved both the provisions to be tested and the test procedures.

OPERATIONS AND SCHEDULE

NBS will commence the activity by inviting each organization to provide representatives to the Committees as listed on attachment 1. NBS will then convene all the Committees for a one-day meeting on December 11. At this time preliminary material will be distributed, the objectives, guidelines, operating policies, and the schedule will be discussed, the committees will select their chairmen, and discussion of the issues for review and refinement will begin. The remainder of this section describes policies and schedules (a schedule is shown on figure 2).

Following the initial meeting, Technical Committee members and other individuals will submit their proposed revisions, in writing and with supporting evidence and reasoning, to the committee secretaries by January 11, 1980. The secretary will then send the proposed revisions to the ATC representative for his review and comment. His review comments will be due back to the secretary by January 30, 1980.

All suggested proposed revisions and ATC responses will be distributed to the committee members prior to the individual committee working meetings scheduled for February. These meetings will constitute the primary opportunity for individual committees to deliberate on the issues, and they may extend over several days. Each Technical Committee secretary will then conduct a letter ballot of his Committee covering all revisions proposed for ballot at the working meetings, and will then make the results available to the members of the Coordinating Committee in advance of their meeting (scheduled for early April). A simple majority will be required to send a proposed revision to the Coordinating Committee.

The Coordinating Committee will identify and resolve all conflicts and inconsistencies created by the proposed revisions of the individual Technical Committees. The Coordinating Committee has the authority to modify the proposal of an individual Technical Committee, to drop any proposal from further consideration, and to combine related proposals from different committees into single proposals. The objective of the Coordinating Committee is to come up with a set of proposed refinements to be discussed at the final meeting of the Joint Committee. Actions of the Coordinating Committee will be carried by a simple majority, except that any individual Technical Committee chairman has the right to place a proposal from his committee that was dropped by the Coordinating Committee on the agenda of the final meeting as a special issue.

Each participant will receive the set of proposed revisions prior to the final Joint Committee meeting (scheduled for May or June). In order to assure each participant's understanding of the ballot issues, each proposed revision will be presented with a concise exposition of pro and con and discussion will be allowed at the final meeting. Upon leaving the final meeting, each voting member will be given a letter ballot, which he is to return within two weeks. Each member will be expected to vote on all proposed revisions. A two-thirds majority of votes cast will be necessary for adoption of any change. NBS will compile the revisions, both proposed and adopted, into a final report with appropriate documentation for any changes suggested to the Tentative Provisions and for the conduct of the trial design and impact assessment activity.

Each of the Technical Committees should examine each proposed provision with regard to the need for early test designs and refer such need to the Coordinating Committee. NBS will work with the Coordinating Committee to expedite carrying out needed designs.

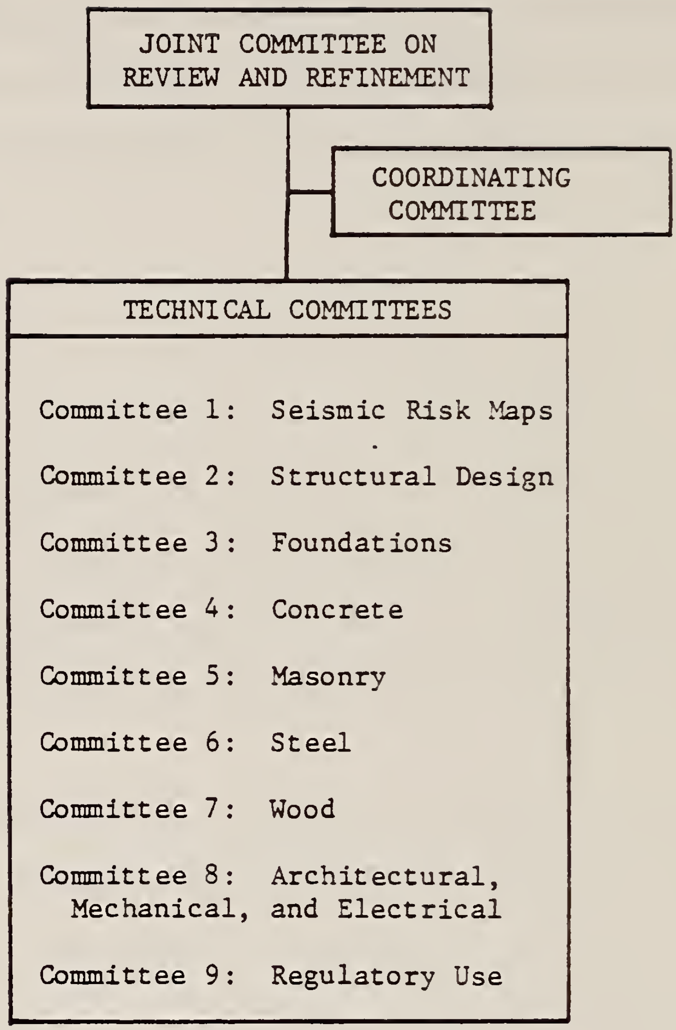

Figure 1: Committee Structure

Figure 2 - SCHEDULE

	1979		1980							
Activity	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug
1. Initial Joint Committee and Technical Committee Meeting		Dec 11								
2. Submission of Proposals		—								
3. Review by ATC			--							
4. Individual Working Meetings				--						
5. Letter Ballots					--					
6. Coordinating Committee						*				
7. Final Joint Meeting							*			
8. Final Ballot							—			
9. Report						----	----	----	----	----0

Attachment 1 - TECHNICAL COMMITTEE MEMBERSHIP

COMMITTEE 1: Seismic Risk Maps

Voting Members

American Society of Civil Engineers
Association of Engineering Geologists
Interagency Committee on Seismic Safety in Construction
Seismological Society of America
Structural Engineers Association of California
United States Geological Survey

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 2: Structural Design

Voting Members

American National Standards Institute
American Society of Civil Engineers
Interagency Committee on Seismic Safety in Construction
Structural Engineers Association of California
Representative from Committee 3 (Foundations)
Representative from Committee 4 (Concrete)
Representative from Committee 5 (Masonry)
Representative from Committee 6 (Steel)
Representative from Committee 7 (Wood)

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 3: Foundations

Voting Members

American Society of Civil Engineers
American Society of Foundation Engineers
Interagency Committee on Seismic Safety in Construction
Structural Engineers Association of California

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 4: Concrete

Voting Members

American Concrete Institute
American Society of Civil Engineers
Interagency Committee on Seismic Safety in Construction
Concrete Reinforcing Steel Institute
Portland Cement Association
Post-Tensioning Institute
Prestressed Concrete Institute

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 5: Masonry

Voting Members

American Concrete Institute
American National Standards Institute
American Society of Civil Engineers
Brick Institute of America
Masonry Institute of America
National Concrete Masonry Association
The Masonry Society
Western States Clay Products Institute

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 6: Steel

Voting Members

American Institute of Steel Construction
American Iron and Steel Institute
American Society of Civil Engineers
Interagency Committee on Seismic Safety in Construction
Metal Building Manufacturers Association
Steel Plate Fabricators Association

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 7: Wood

Voting Members

American Institute of Timber Construction
American Plywood Association
American Society of Civil Engineers
Interagency Committee on Seismic Safety in Construction
National Forest Products Association
United States Forest Product Laboratory

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 8: Architectural, Mechanical, and Electrical

Voting Members

American Institute of Architects
American Society of Civil Engineers
American Society of Heating, Refrigerating and Air Conditioning Engineers
American Society of Mechanical Engineers
American Society of Plumbing Engineers
Brick Institute of America
Institute of Electrical and Electronics Engineers
Interagency Committee on Seismic Safety in Construction
National Elevator Industry, Inc.
National Fire Protection Association

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

COMMITTEE 9: Regulatory Use

Voting Members

American Institute of Architects
American Insurance Association
American Society of Civil Engineers
Associated General Contractors of America
Association of Major City Building Officials
Building Officials and Code Administrators International
Building Owners and Managers Association
Construction Research Council
Construction Specifications Institute
International Association of Electrical Inspectors
International Association of Plumbing and Mechanical Officials
International Conference of Building Officials
National Conference of States on Building Codes and Standards

National Electrical Manufacturers Association
Southern Building Code Congress International

Nonvoting Members

ATC Representative
BSSC Overview
NBS Secretariat

Attachment 2 - "Objectives" and "New Concepts"

The following "OBJECTIVES" and "NEW CONCEPTS" are quoted directly from the ATC-3 document as an aid in establishing guidelines for the proposed review and refinement.

OBJECTIVES

During the early stages of the ATC-3 project, much time and effort was expended to establish the project objectives. For guidance in understanding the project goals and this document, the objectives are listed below:

1. To evaluate the knowledge acquired in recent research and experience gained during on-site observations of the effects of earthquakes and to assemble it in a concise and comprehensive document for general use by building design professionals and others.

2. To write the tentative design provisions so as to permit, insofar as possible, ingenuity of solution, but with definitive criteria to evaluate the resulting design.

3. To provide seismic protection criteria which will be applicable to all probable earthquake areas of the United States.

4. To recognize that acceptable seismic risk is a matter of public policy determined by a specific government body and should be based upon:

   (a) An evaluation of available technical knowledge, including the areas of seismicity.

   (b) Reasonable means available for protection.

   (c) The magnitude of the earthquake risk compared with acceptable risks for other hazards.

   (d) The economical and social impact of a major catastrophe.

5. To provide tentative design provisions applicable to all buildings, including existing buildings, and appropriate structural as well as nonstructural components. To include requirements for structural analysis, design, and detailing which will provide adequate earthquake resistance for typical buildings and to make recommendations with respect to the design of atypical buildings.

6. To recognize that for critical facilities there should be consideration in the design of building structural and non-structural systems of limiting damage in order to maintain the level of function determined to be necessary.

7. To provide a commentary to assist the user in understanding the intent and background of the provisions and to assess the

implications of any alterations made to the provisions in the future.

NEW CONCEPTS

The provisions embody several new concepts listed below which are significant departures from existing seismic design provisions. Consequently, the provisions should not be considered for code adoption until a detailed evaluation is made of their workability, practicability, and potential economic impact.

1. The incorporation of more realistic seismic ground motion intensities.

2. Consideration of the effects of distant earthquakes on long-period buildings.

3. Response modification coefficients (reduction factors) which are based on consideration of the inherent toughness, amount of damping when undergoing inelastic response, and observed past performance of various types of framing systems.

4. Classification of building use-group categories into "Seismic Hazard Exposure Group."

5. Seismic performance categories for buildings with design and analysis requirements dependent on the seismicity index and building seismic hazard exposure group.

6. Simplified structural response coefficient formulas related to the fundamental period of the seismic resisting system of the building.

7. Detailed seismic design requirements for architectural, electrical, and mechanical systems and components.

8. Materials design and analysis based upon stresses approach yield.

9. Guidelines for systematic abatement of seismic hazards in existing buildings.

10. Guidelines for assessment of earthquake damage, strengthening or repair of damaged buildings, and potential seismic hazards in existing buildings.

APPENDIX E

ROSTER OF PARTICIPANTS

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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)

The Tentative Provisions for the Development of Seismic Regulations for Buildings were developed by the Applied Technology Council to present, in one comprehensive document, current state-of-knowledge pertaining to seismic engineering of buildings. The Tentative Provisions are in the process of being assessed by the building community. This report is one of a series of reports that documents the deliberations of a group of professionals jointly selected by the Building Seismic Safety Council and the National Bureau of Standards and charged with reviewing the Tentative Provisions prior to the conduct of trial designs. The group is divided into nine technical committees, each of which focused on a particular portion of the Tentative Provisions. The nine committees proposed recommendations for change to the parent group, the Joint Committee, through a Coordinating Committee. The Coordinating Committee made some modifications to the technical committees' recommendations to ensure consistency among the recommendations. This report documents the actions of the Joint Committee on the 198 recommendations for change that were presented to it. The first part of the report is a summary of the results, and the appendices contain the full documentation for each recommended change. The actions of each of the nine technical committees is documented in a separate report.

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