NBSIR 2111-4

REVIEW AND REFINEMENT OF ATC 3-06 TENTATIVE SEISMIC PROVISIONS

REPORT OF TECHNICAL COMMITTEE #4: CONCRETE

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Abstract

The TENTATIVE PROVISIONS FOR THE DEVELOPMENT OF SEISMIC REGULATIONS FOR BUILDING 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 beign 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 report contains the recommendations and records of the committee charged with review of the reinforced concrete design provisions. The committee made 19 recommendations for revisions to the TENTATIVE PROVISIONS. These recommendations were made to the parent group, the Joint Committee on Review and Refinement, and their action on these recommendations is documented in a companion report.

Key Words: Building; building codes; building design; earthquakes; engineering; reinforced concrete; standards; structural engineering.

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1.0 INTRODUCTION

1.1 General

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 with the advice and approval of the Building Seismic Safety Council, 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 assessment of the Tentative Provisions was performed using the committee structure shown in figure 1. 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 the committee membership section 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 (ATC), the Building Seismic Safety Council (BSSC) and the National Bureau of Standards (NBS). 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.

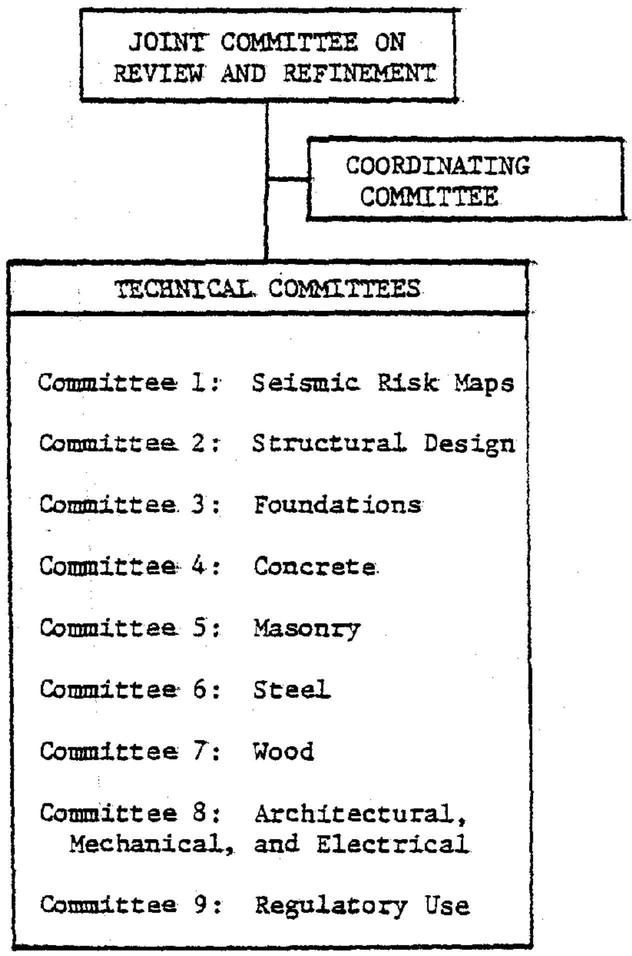

Figure 1: Committee Structure

1.2 Committee Summary

Technical Committee 4 had as its principal responsibility the review and refinement of the provisions in Chapter 11, Reinforced Concrete, of the Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC3-06). The committee membership was drawn from industry, professional organizations, and standards development organizations. The committee conducted four meetings. Two meetings, the first and fourth, were held in Gaithersburg, Maryland. The second meeting was held in San Francisco, California and the third meeting was conducted in Skokie, Illinois. Following is a brief summary of the committee actions during each meeting and a summary of the overall direction of the committee.

The first meeting was a half day meeting held on December 11, 1979, in Gaithersburg, Maryland. The committee chairman, Mr. Cohen, was elected and Mr. Fintel was elected to serve as the committee's representative on Technical Committee 2 - Structural Design. The committee expressed its intent to concentrate on the provisions of Chapter 11 and to also consider changes to provisions in other chapters. Prepared comments from Mr. Sheppard representing the Prestressed Concrete Institute and Mr. Fintel representing the Portland Cement Association were received and reviewed by the committee. The comments by Mr. Sheppard generally addressed the omission of specific mention and requirements for precast and/or prestressed concrete in the ATC3-06 provisions. The restriction against the use of precast-prestressed piles in Seismic Performance Category D buildings was also discussed by Mr. Sheppard. The thrust of Mr. Fintel's comments was a basic objection to the manner in which the seismic response modification coefficient (R) and the deflection amplification factor (C_d) were determined. The meeting adjourned without finishing the discussion of the prepared comments.

The second meeting was held in San Francisco, California on February 21, 1980. The meeting lasted for a full day and evening. The meeting was announced in several national professional publications and the announcement called attention to the fact that the meeting was open to any interested party. A major issue addressed in the meeting was a proposal to adopt the 19 March 1980 draft version of ACI 318 Appendix A in lieu of the ATC3-06 Chapter 11. After considerable discussion the committee agreed to ask for guidance from the Coordinating Committee before taking action on the proposal. The committee was unsure that such a major change was within the scope of the committee's task. The committee adopted the position that it would continue to revise ATC3-06 Chapter 11 while waiting for guidance from the Coordinating Committee. The committee then discussed the proposed changes to ATC3-06 submitted by Mr. Sheppard, Mr. Fintel, Mr. Manning, and Mr. Forrell. The committee decided by voice vote which changes would appear on a committee letter ballot. The committee adjourned the meeting with some proposed changes requiring further discussion, but decided before adjournment to issue a letter ballot containing the agreed upon recommendations.

A letter ballot (dated March 27, 1980) containing six proposed changes to ATC3-06 Chapter 11 and nine proposed changes to other chapters was distributed to the committee between the second and third meetings. Some committee members failed to return their ballot in time for the results to be discussed at the third meeting. As a result, the resolution of "no" and "yes with reservations" votes could not be completed at the third meeting.

The third meeting of Committee 4 was held for a full day in Skokie, Illinois on April 14, 1980. The first issue that was discussed was the proposed adoption of the 19 March 1980 draft version of ACI 318 Appendix A in lieu of ATC3-06 Chapter 11. In response to concerns about the compatibility between the remainder of ATC3-06 and Appendix A, and at the request of the committee, Mr. Neville, ACI Committee 318 secretary, prepared a new Chapter 11 (hereinafter called Revised Chapter 11) to serve as a transition chapter between ATC3-06 and Appendix A. The Revised Chapter 11 used ACI 318-77 "Building Code Requirements for Reinforced Concrete" and the 19 March 1980 draft version of Appendix A "Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions" as cited references. Considerable discussion by the committee resulted in a compromise between making changes to the ATC3-06 Chapter 11 and replacing ATC3-06 Chapter 11 with the Revised Chapter 11. The compromise was to cite the 19 March 1980 draft Appendix A in the reference of the ATC3-06 Chapter 11 and make the recommended changes to ATC3-06 Chapter 11. The compromise was placed on a letter ballot (the second letter ballot, dated May 5, 1980). The committee then continued discussion of the proposed changes remaining from the second meeting and discussed proposed changes concerning flat slab systems submitted by Mr. Hawkins. The committee decided by voice vote which proposed changes would appear on a letter ballot.

The second letter ballot (dated May 5, 1980) was prepared and distributed to the committee after the third meeting and prior to the fourth meeting which was called at the request of the voting members to discuss the issue of the Revised Chapter 11 once again.

The fourth and final meeting was held in Gaithersburg, Maryland on June 4, 1980. The meeting lasted a full day and evening. The committee took up the resolution of "no" and "yes with reservation" votes on the two letter ballots as the first order of business. The process was tabled to consider the adoption of the Revised Chapter 11 in lieu of the original Chapter 11 in ATC3-06. After vigorous discussion on the appropriateness of making such a change, the committee voted by a show of hands to adopt the Revised Chapter 11. The committee then completed action on the letter ballot items. The committee reviewed specific objections to the Revised Chapter 11 and the 19 March 1980 draft Appendix A in an attempt to resolve the objections. As a final action, the committee prepared letter ballot items for issues raised in the meeting and conducted the ballot.

Technical Committee 4 began its work by considering changes to the existing ATC3-06 provisions, both in Chapters 11 and others. The direction changed such that the final committee position was to recommend a completely new Chapter 11 which cited the 19 March 1980 draft version of ACI 318 Appendix A as the reference for the principal technical provisions. It was clear that the committee was firmly resolved to incorporate the latest version of Appendix A in Chapter 11 because it represented the state-of-the-art in reinforced concrete seismic provisions. Other major issues endorsed by

Committee 4 were provisions for the design of flat slab systems in Seismic Performance Category B buildings, the inclusion of a clause to permit any system which could be analytically and experimentally demonstrated to have characteristics similar to a comparable monolithic cast-in-place reinforced concrete system, and the use of precast- prestressed piles in Seismic Performance Categories C and D buildings.

Those members of Committee 4 present in Gaithersburg, Maryland for the Joint Committee Meeting (July 16-17, 1980) met on the afternoon of July 16. The meeting was informal and impromptu. The discussion in the meeting centered on the Joint Committee's reaction to the revised Chapter 11. It was generally agreed that the presentation of the all inclusive ballot item was not as desirable as a detailed breakdown. The committee members present agreed that if the inclusive ballot item were to fail, Committee 4 should request that the Building Seismic Safety Council permit a restructuring of the ballot item and its submission to the Building Seismic Safety Council for balloting as part of its own letter ballot.

1.3 Chairman's Statement

As set forth in the work plan for review and refinement of ATC3-06, Technical Committee 4's primary responsibility was Chapter 11 - Reinforced Concrete. The committe made an in-depth review of the chapter, particularly with respect to impending action within American Concrete Institute's (ACI's) Building Code Committee 318. The Committee action was to include the latest ACI seismic provisions in the ATC document. The Committee attempted to compare the design provisions of Chapter 11 with the more recently developed ACI provisions and realized that numerous changes would be necessary to upgrade existing Chapter 11 to the latest ACI criteria. Thus, Committee 4 determined to recommend adoption of the new ACI provisions for earthquake resistance into ATC, considering this to be the most efficient approach. (Two notable examples of more recent developments contained in the ACI criteria are (1) anchorage length for reinforcement provisions in ACI are upgraded, based on new experimental data and reevaluation of all previous data, and (2) design of joints of frames is upgraded to reflect the latest report of the ACI-ASCE Joint Committee 352).

Therefore, with respect to Chapter 11, Committee 4 is recommending 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 ATC3-06 for proportioning and detailing concrete structures. (Revised Appendix A, dated 19 March 1980, is to supercede Appendix A of ACI 318-77).

Revised Appendix A is now before the full ACI Building Code Committee 318. Final Committee action and full ACI consensus balloting is forthcoming. Considering that the primary mission in development of ATC3-06 is to provide the most current state-of-knowledge for seismic design, Committee 4 considers it prudent to use the latest seismic proportioning provisions for the trial design phase of the ATC review process. Should further revision occur in the ACI seismic design provisions between now and final ACI adoption of new Appendix A, appropriate cross reference correction can be made in Chapter 11 of ATC. It is however, the intent of Committee 4 that new Appendix A, dated 19 March 1980, be used in the trial design phase.

Committee 4 also adopted unanimously 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 ATC3-06 after trial design."

Justification for the above recommendation is outlined as follows for consideration by the Joint Committee:

1. Adoption of the total ACI 318 Standard is considered appropriate because seismic resistance is considered in the overall development of the 318 Standard, including Appendix A on special provisions for earthquake resistance.

2. 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.

3. 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.

4. 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.

Adoption of the ACI 318 Standard, including new Appendix A, into ATC3-06 necessitated a complete revision of Chapter 11. The following new Chapter 11 has been formulated to correlate appropriate ACI 318 design provisions with the four ATC seismic performance categories by reference only, without the need for duplicating in the ATC document the wording already contained in the ACI document. New Chapter 11 specifies where the design provisions of ACI 318 apply for seismic resistance within the framework of the established ATC seismic analysis and performance criteria. Further, new Chapter 11 includes special provisions for flat plate framing systems for buildings assigned to Category B and special consideration of precast/prestressed framing systems. These two items are discussed in the following attachment.

In summary, existing ATC Chapter 11 is "technically" updated to the new Appendix provisions of the ACI 318 Standard.

Under these circumstances, and since the ACI 318 Standard is a fully approved consensus document, to avoid overlapping and conflicting efforts and criteria, and to assure that Chapter 11 represents the highest practical state-of-knowledge, Committee 4 strongly recommends that, in the national interest, full adoption of ACI 318 Standard, by reference, is approved.

The Committee wishes to thank Messrs. V. V. Bertero and James Lefter for their dedication and many technical contributions to the work of the Committee over the past months, and in addition, special thanks to Committee Secretaries R. Marshall and K. Woodward for their support of the Committee work.

ATTACHMENT

In addition to adoption of ACI 318-77, and the revised Appendix A, Committee 4 is recommending two exceptions in Chapter 11:

1. SPECIAL PROVISIONS FOR FLAT PLATE FRAMING SYSTEMS FOR BUILDINGS ASSIGNED TO CATEGORY B, and
2. SPECIAL CONSIDERATION OF PRECAST/PRESTRESSED FRAMING SYSTEMS.

Both items are included in Chapter 11 as EXCEPTIONS. The flat plate framing exception is under Sec. 11.4.1. The EXCEPTION applies only for flat plate framing systems assigned to Category B. Explanation of the special provisions is given in Sec. 11.4. on page 3 of new Commentary to Chapter 11. The precast/prestressed exception is in Sec. 11.4.2 for Category B and Sec. 11.5.2 for Categories C and D. The exception refers to systems that are shown to meet the performance requirements (strength, toughness, ductility, etc.) of monolithic cast-in-place reinforced concrete structures or, alternatively, are proportioned for acceptable higher lateral forces to remain elastic under earthquake loadings. The Commentary emphasizes that precast and/or prestressed concrete elements and assemblages may be used to meet either of the above requirements, which is similar to the situation under which precast and/or prestressed concrete structures are currently designed and built under the Uniform Building Code. Documentation for the flat plate exception follows:

SPECIAL PROVISIONS FOR FLAT PLATE FRAMING SYSTEMS FOR BUILDINGS ASSIGNED TO CATEGORY B.

1. <u>Implication of ATC 3-06 if unamended for flat plate construction in seismic performance Category B</u>.

The intention of ATC 3 with respect to restrictions on the use of flat plate framing to resist lateral forces for Category C and D structures is clear. Such use is highly undesirable. However, for buildings in seismic hazard exposure Category B, ATC 3-06 also effectively prohibits flat plate construction. Category B includes all facilities in New York, Boston, Buffalo, and the New England states, much of North and South Carolina and Tennessee, large areas of Oklahoma, Arkansas, Illinois, and Missouri, and much of New Mexico, Montana, Idaho, Utah, Washington and Wyoming. The potential economic impact of that prohibition is staggering. For the post-tensioning industry alone, that prohibition could mean about a 25% drop in its work volume. Post-tensioning is used in about 30 million square feet of suspended slabs constructed each year in the U.S.A. and much of that construction is flat plate.

In his letter of November 19, 1979, to ATC, Jacob Grossman of Robert Rosenwasser and Associates of New York, details his experience with respect to the economics of flat plate construction and its seismic response when properly detailed. He states "I cannot even begin to describe the construction havoc the exclusion of flat-slab structures can introduce in strong union-high construction cost areas." He points out that enough research and knowledge are available to incorporate flat-slabs and allow them as "ordinary" frames without shear reinforcing.

It is not clear that it was intended that ATC 3-06 prohibit flat-slab construction for "ordinary" frames. Contrary to the situation for Category C and D exposures, neither the provisions nore the Commentary explicitly state such a prohibition. However, they require in flexural members of ordinary frames for Category B exposure, web reinforcement perpendicular to the longitudinal reinforcement throughout the length of the member. The minimum reinforcement is two leg No. 3 stirrups at a spacing of d/2. Inclusion of such reinforcement in flat slabs would create economic as well as logistic problems.

It is suggested that:

(a) the provisions state clearly whether flat plate, flat slab, or waffle slab construction is feasible for ordinary frames for Category B;

(b) if flat plate, flat slab, or waffle slab construction is feasible, the provisions specify any special restrictions for that construction.

2. Behavior of flat plate construction under cyclic loading

2.1 Laboratory results

There have been seven major laboratory investigations of flat plate connections subjected to cyclic loading (1-8). The results of extensive University of Washington investigations are summarized in the attached articles.

The lateral load response is strongly influenced by:

(a) the amount and distribution of the flexural reinforcement in the slab,

(b) the amount, type and extent of any shear reinforcement, and

(c) the level of shear stress transferred to the column simultaneous with the moment.

Even when there has been a low flexural reinforcement ratio and a connection well over-designed for shear, there has still been little ductility under reversed cyclic loading. For specimens with high reinforcement ratios within lines one and one-half times the slab thickness either side of the column, there has been a considerable increase in the lateral load stiffness. However, there has been as much as a 10% reduction in the moment transfer capacity as compared to that for monotonic loading and a punching failure has occurred shortly after the reinforcement passing through the column has yielded. Since there is little improvement in the ductility with the use of low reinforcement ratios and a considerable reduction in stiffness, concentration of column strip reinforcement is desirable.

The only proven ways of maintaining capacity through large rotations has been to add properly detailed shear reinforcement consisting of either integral beam stirrups or thin steel H sections or studs anchored above and below the flexural reinforcement passing through the column. Shear reinforcement in the form of shearheads or bent bars increases the capacity but does not increase ductility. The shear reinforcement must hold the top and bottom flexural mats together and prevent the development of a splitting crack between those mats. The shear reinforcement should have a spacing not exceeding d/2 and need not extend further than about 5 slab thicknesses out from each column face. Rules for proper detailing of such shear reinforcement are described in Reference 7. Slab-column connections are so flexible that flat plate structures are unlikely to meet ATC 3-06's stiffness requirements for ductile moment resistant frames. Thus, shear reinforcement in flat plates is probably unnecessary unless the flat plate structure is the only line of defense or unless the flat plate structure is to provide a required secondary line of defense.

The level of shear stress transferred simultaneously with the moment markedly affects the energy dissipation and ductility characteristics of slab-column connections. To obtain desirable characteristics, the flexural reinforcement within lines one and one-half times the slab thicknesses, either side of the column should be limited to one percent and the shear stress due to shear transfer on the critical section d/2 from the column perimeter to $3\sqrt{f'_c}$. At that latter stress, shear cracks have not developed in the slab prior to the application of lateral loading.

After a punching failure has occurred, bottom bar flexural reinforcement continuous through the column is essential to the connection being able to maintain its gravity load carrying capacity. Such reinforcement can carry a shear force equal to its shearing yield capacity. Alternatively, prestressing reinforcement passing through a column or over a lift slab collar is also a very effective means of tying a slab-column connection together after a punching failure. With prestressing reinforcement, a residual capacity can be obtained equal to 90 percent of the pre-punching shear capacity.

Shear or torsional cracking develops at the discontinuous edge of a slab adjacent to an exterior column, when the shear stress at that location evaluated according to ACI 318-77 provisions, exceeds $3\sqrt{f'_c}$. If that stress is exceeded, stirrups having a size not less than No. 3, a spacing equal to or less than d/2, and extending up to four times the slab thickness from the torsional faces of the column should be provided to prevent opening of those cracks. While the best ductility and energy dissipation characteristics are obtained with integral beam stirrups, hairpin stirrups inserted perpendicular to the edge and extending a distance equal to the column projection into the slab plus ℓ_d, or twice the slab thickness plus ℓ_d, whichever is less, into the slab will also provide adequate control to the opening of those cracks.

Tests have shown that the above results are also applicable to waffle slab-interior column connections (4) and that when there is moment transfer about both axes of a column (8), the effect of the minor moment on the shear capacity can be neglected if that moment does not exceed 30% of the major moment and there is adequate reinforcement within lines one and one-half times the slab thickness either side of the column to transfer that minor moment.

The recent tests on flat plate frames at the University of Washington (8) have shown that in a frame, a punching failure will occur at a connection without stirrups at a displacement and at a capacity consistent with subassemblage results. However, that punching failure did not lead to an immediate loss in capacity of the overall frame since the adjacent connection with shear reinforcement was able to supply the required additional moment transfer capacity. Displacements much greater than those for punching at the connection without shear reinforcement had to be applied before the capacity of the overall frame deteriorated. During that period the connection without shear reinforcement continued to carry its share of the gravity load. Further, after lateral loading was completed, it was found that at the punched connection the slab could be readily jacked back up to its original elevation and the connection repaired.

2.2 Field results

Reports (9) have been issued on the behavior of several flat plate structures in the San Fernando earthquake; the Holiday Inn, Orion Avenue, the Holiday Inn, Marengo, and the Muir Medical Center. The Holiday Inn, Orion Avenue, was a seven-story reinforced concrete flat plate structure with typical plan dimensions 62 by 160 ft. The building was supported on piles centered under the columns which were spaced at approximately 20 ft. centers. Spandrel beams approximately 16 x22½ in. surrounded the perimeter of the structure. The flat plate floor was 10 in. thick at the second floor, 8 in. thick at the roof, and 8½ in. thick for all other floors. The spandrel beams were figured as creating exterior frames roughly twice as stiff as the interior flat plates so that in the short and long directions of the building, 36% and 67%, respectively, of the stiffness was provided by the exterior frames. Peak accelerations at the first floor level were 0.25g and 0.13g in the short and long directions, respectively. Roof accelerations were 0.41 and 0.33g, respectively. Repairs cost 11% of the initial construction cost and were nearly all nonstructural. Some structural distress occurred at the corner column beam connections and in the construction joints at the soffit of the exterior column-beam connections. The response was most marked in the short direction with a lengthening of the period part way through the record indicating that the structure began responding inelastically. The analysis indicated that beams and slabs yielded, that columns generally remained elastic, and that interstory drifts as large as 0.13 ft. occurred. The elastic limit displacement was roughly 2½ times the design code displacement.

The Holiday Inn, Marengo Street, had dimensions and member sizes almost exactly the same as the Orion Avenue building. Peak accelerations at the first floor level were 0.15g and 0.14g in the short and long directions, respectively. Roof accelerations were 0.43g and 0.25g, respectively. Repairs cost 7% of the initial construction cost and were nearly all non-structural. Structural distress was similar to that for the Orion Avenue building. The dynamic response was also similar to the Orion Avenue building. The analysis indicated that heams and slabs yielded at their connections with the columns but that the columns generally remained elastic. Interstory drifts were as large as 0.14 ft. In this structure, as in the Orion Avenue structure, it was apparent that the stiffness of the frame was sufficiently low that the non-structural elements such as partitions, played an appreciable role in the character of the structural response to seismic forces.

The Muir Medical Center was an 11-story office tower with an 18½ in. deep waffle slab at the ground level and perimeter basement walls. For the second floor and above, the 9-in. thick flat slab had 15 in. deep tapered column capitals with deep spandrel beams around the perimeter. The deep spandrel beams framing provided 70% of the lateral load stiffness and the interior flat-slab-tapered drop panel framing the other 30%. Peak accelerations were 0.10g at the basement level and 0.2g at the roof level. Some of the structural members were predicted to yield during the earthquake and the maximum story drift was computed to be 0.64 in. The general performance of the structure was linear-elastic with only minor lengthening of the building period during the earthquake. Damage was all non-structural and estimated at less than $2,000.

2.3 Period of Vibration

Fundamental periods for those structures for man-induced excitations prior to the earthquake, at the beginning of the earthquake, mid-way through the earthquake, and for man-induced excitations after the earthquake are listed in Table 1. It is apparent that the period of the predominantly flat plate structures, the Holiday Inns, increased noticeably during the earthquakes with the increase being larger for the more heavily shaken Orion Avenue building.

TABLE 1
Periods of Vibration for Flat Slab Structures

	Stories	Direction	Periods			
			Before Quake	Start of Quake	Midway Thru Quake	After Quake
Holiday Inn Orion Ave.	7	short	0.48	0.79	1.6	0.68
		long	0.53	0.88	1.24	0.72
Holiday Inn Marengo	7	long	0.53	0.88	1.0	0.64
		short	0.49	0.79	1.2	0.63
Muir Medical	11	long	0.90	1.43	1.4	1.02
		short	1.03	1.60	1.6	1.14

If the values of the periods at the start of the earthquake are compared with the general data on Fig. C4-2, page 373 of ATC 3-06, then it is apparent that the periods for these structures are better characterized by the relationship, $T_R = 0.035h_n^{3/4}$ than the relationship $T_R = 0.025h_n^{3/4}$.

2.4 Stiffness

The ATC 3-06 provisions limit the allowable story drift for Seismic Hazards Exposure Groups I and II to 0.015 radians. When there are no brittle-type finishes in buildings three stories or less in height, those values can be increased to 0.02 radians. If it is accepted that for a reinforced concrete structure, a load factor of 1.4 is required on earthquake forces and a capacity reduction factor of 0.9 for flexure, then connection rotations at 65% of the ultimate capacity should not exceed the story drift specified above divided by C_d. Maximum measured values of C_d for a story drift of say 0.015 radians for a given column proportion and spacing can be evaluated directly from the subassemblage specimens.

Table 2 lists C_d values calculated according to that concept for several different investigations. Values range from a low of 2.4 for the flat plate frame test (8) with a low ρ value through to a high of 4.3 for the waffle slab specimen (4). There is a marked increase in values with increasing slab depth and a lesser increase with increasing column size. Not apparent from that table is the wide variation in results obtained for supposedly identical specimens. C_d values varied by as much as 50% for similar specimens and averaged about 20% higher for specimens with shear reinforcement than those without.

Based on these subassemblage results and experience from the San Fernando earthquake, it is apparent that a conservative value of C_d for flat plate structures is 2. Although higher values can be obtained by careful detailing, even for waffle slabs, it is unrealistic to expect that the C_d value of 6 required for a ductile moment resistant frame can be obtained. Thus, flat plate framing should only be recognized as an acceptable lateral load resisting system when classified as an ordinary frame. The only possible exception might be for a waffle slab structure without brittle finishes and less than three stories high. Even in that case, experience from the San Fernando earthquake with the Olive View Hospital

Ambulance Port was undesirable. However, the 14 x 18 in. columns were smaller than desirable and failure occurred in the columns and not in the slab-column connection.

2.5 Conclusions

Based on this summary of field and laboratory experience, it is concluded that:

(1) flat plate structures of normal proportions and without shear reinforcement will have little difficulty in meeting the strength, stiffness, ductility, etc., requirements for ordinary frames, especially if certain detailing requirements specified later, are satisfied.

(2) with flat plate structures of normal proportions it would be extremely difficult, if not impossible, to meet the stiffness requirements for utilizing such frames as special moment frames Category C and D buildings.

(3) with flat plates of normal proportions punching failures will not occur until interstory drifts greater than the limiting values specified in Table 3-C, page 53, of ATC 3-06.

(4) with flat plate structures used as the gravity load carrying system in Category C and D buildings, it is not necessary to consider punching failures as unacceptable provided the detailing requirements, specified later, are satisfied.

(5) with flat plate structures yielding should be defined as either:

(a) the development of the negative moment yield capacity of the slab on a line extending across the width of the slab at the column face, or

(b) the development of the moment transfer capacity at the slab-column connection for yielding of the reinforcement at that connection. That capacity can be taken as the flexural capacity of the reinforcement top and bottom within lines one and one-half times the slab thickness either side of the column.

(6) the period of structures with 35% or more of the lateral load stiffness provided by flat plate framing can be estimated from the relationship $T_R = 0.035h^{3/4}$.

Attachment - 9

TABLE 2

VALUES OF C_d MEASURED IN SUB-ASSEMBLAGE TESTS

Reference	Scale	Full Scale Properties: Slab Thickness in.	Column Spacing ft.	Column Size in. x in.	Story Height ft.	C_d	Specimen Type
1	3/8	8	18	16 x 16	13	2.5	Flat Plate
2	Full	7.5	19	18 x 18	11	2.5	Flat Plate
3	0.4	10	20	20 x 20	10	3.5	Flat Plate
4	1/4	14	20	20 x 20	11	4.3	Waffle Slab
7	3/4	8	16	21 x 21	11	2.8	Flat Plate
8	1/2	9	24	19½ x 8	9	2.4	Flat Plate Frame-lowo

References

(1) Hanson, N. and Hanson, J., "Shear and Moment Transfer Between Concrete Slabs and Columns," Journal of the PCA Research and Development Laboratories, January 1968. $C_d = 2.5$

(2) Carpenter, J.E., Kaar, P.H., and Corley, W.G., "Design of Ductile Flat Plate Structures to Resist Earthquakes," Proc. 5th World Conf. Earthquake Eng., Rome, Italy, 1973.

(3) Kanoh, Y. and Yoshinzaki, S., "Experiments on Slab-to-Column and Slab-to-Wall Connections," Japan Concrete Journal, Vol. 13, No. 6, June 1975 $C_d = 3.5$

(4) Rodriguez, M.R., "Diseno Sismico De Conexiones Entre Losas Planas Reticulares y Columnas," M.E. Thesis, University of Mexico, July 1979. $C_d = 4.3$

(5) Islam, S. and Park, R., "Tests on Slab-Column Connections with Shear and Unbalanced Flexure," Journal of the Structural Division, ASCE, Vol. 102, No. ST3, March 1976, pp.549-568.

(6) Zaghlool, E.E.R., "Strength and Behavior of Corner and Edge Column-Slab Connections in Reinforced Concrete Flat Plates," Ph.D. Thesis, University of Calgary, Alberta, Canada, 1971.

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

(8) Hawkins, N.M., "Seismic Response of Concrete Flat Plate Structures," Proc. Seventh World Conference on Earthquake Engineering, Istanbul, 1980.

(9) San Fernando, California, Earthquake of February 9, 1971, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1973.

2.0 COMMITTEE ACTIONS

2.1 Recommendations for Change

2.2 Recommendations for Trial Designs (none)

2.3 Recommendations for Commentary

2.4 Other Recommendations (none)

2.1 Recommendations for Change

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: 86(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 0.2Ø 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

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

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: 87

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 3-8. 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: YI

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 Code 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.

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:

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

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."

3.0 COMMITTEE RECORDS

3.1 Minutes of Meetings

Minutes of First Meeting

Technical Committee 4: Concrete

Review and Refinement of Tentative

Seismic Provisions (ATC 3-06)

Meeting Held at National Bureau of Standards

Gaithersburg, Maryland

December 11, 1979

The first meeting of Technical Committee 4 was called to order at 12:10 p.m. by Acting Chairman Richard D. Marshall. The following members were present:

Name	Representative of
Edward Cohen	American Concrete Institute
Mark Fintel	Portland Cement Association
Neil M. Hawkins	Post-Tensioning Institute
Joseph Manning	Concrete Reinforcing Steel Institute
James Prendergast	Interagency Committee on Seismic Safety in Construction
David A. Sheppard	Prestressed Concrete Institute
W. Gene Corley (alternate)	Portland Cement Association
Vitelmo V. Bertero	Applied Technology Council
James Lefter	Building Seismic Safety Council
Richard Marshall	National Bureau of Standards

The first order of business was the selection of a permanent committee chairman. Edward Cohen was nominated and unanimously elected. Mr. Cohen chaired the subsequent committee deliberations.

The next item of business was the selection of an individual to serve on Technical Committee 2 - Structural Design. Mark Fintel was nominated and unanimously elected.

Time and place of the second meeting (public work session) was the next item of business. This meeting is to be announced in selected publications having national circulation. After considerable discussion, it was decided that the meeting would be held on February 21 (and 22, if necessary), 1980 at the Airport Hilton Hotel, San Francisco, California. It was agreed that the Committee Secretary, R. D. Marshall, would make arrangements for a conference room.

The committee broke for lunch at 1:00 p.m. and reconvened at 2:00 p.m.

The afternoon session opened with general discussion as to what sections of ATC 3-06 were to be reviewed by Committee 4 and what procedures were to be followed in developing and submitting proposed changes. In addition to the provisions of Chapter 11 - Reinforced Concrete, concern was expressed regarding certain provisions of Chapters 3, 4, 5, 7 and 8. Edward Cohen inquired as to the status of refinements to the provisions related to reinforced concrete which are now being carried out by the Interagency Committee on Seismic Safety in Construction. James Lefter, representative of the BSSC Overview Committee, stated that he would provide the secretary with the latest draft for distribution to members of Technical Committee 4.

With regard to proposed changes, it was determined that individual members may submit proposed changes directly to the appropriate technical committees with copies to the membership of Committee 4. Also, it was agreed that proposals may be developed jointly by the membership of Committee 4, for consideration by other technical committees.

The chairman next asked for specific comments on the current provisions of ATC 3-06. Prepared comments by the Prestressed Concrete Institute were distributed to the membership (see Attachment A) by David Sheppard and were discussed at length. Regarding the provisions of Chapter 7, it was pointed out by the chairman that foundation design criteria should be the responsibility of Technical Committee 3, but that material-specific design provisions should be presented in Chapters 10 and 11 (steel and reinforced concrete).

Mark Fintel also referred to prepared comments (see Attachment B) in addressing the relevant provisions of ATC 3-06. The issue of Response Modification Coefficients, R, was discussed at some length. Fintel pointed out that while he feels the general approach is a significant step forward, the R values (Table 3-B) are based primarily on the judgment of a group of individuals rather than on rational analysis. To rectify this situation, Fintel proposed that studies of dynamic, inelastic response history be carried out for various structural systems identified in Table 3-B. V. Bertero stated that while the studies proposed by Fintel would be very useful, they would, in his opinion, require considerable effort and would quite likely require more than one year to complete. Joseph Manning suggested that additional insight regarding the response modification coefficients could be obtained during the trial design process by subjecting a building to both elastic and inelastic analysis. V. Bertero stated that additional information considered by the Applied Technology Council, but not contained in Table 3-B, could be made available to the members of Committee 4.

Neil Hawkins pointed out that the current provisions allow the substitution of grade 60 for ASTM A-615 grade 40 reinforcement which could, in certain cases, be detrimental.

Because of tight travel schedules, the committee adjourned at 4:00 p.m. without completing discussion of comments by individual members.

Respectfully submitted,

Richard D. Marshall

Richard D. Marshall

ATTACHMENT A

ARCHITECTURAL
STRUCTURAL
PCI PRESTRESSED CONCRETE INSTITUTE®

REPLY TO:
1350 DEL RIO COURT
CONCORD, CALIFORNIA 94518

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20 NORTH WACKER DRIVE / CHICAGO, ILLINOIS 60606

TELEPHONE 312 / 346-4071

November 5, 1979

William W. Moore, Chairman
Board of Direction
Building Seismic Safety Council
Dames and Moore
500 Sansome Street
San Francisco, CA 94111

Re: Constructive comments on ATC 3-06 regarding the design of precast and prestressed concrete under seismic conditions.

Dear Mr. Moore:

The Prestressed Concrete Institute is an organization devoted to the development and advancement of knowledge and use of plant cast precast and prestressed concrete. To that end, we are appreciative of this opportunity to work with our peers on the Building Seismic Safety Council in the critique and review of ATC 3-06 as it pertains to precast and prestressed concrete. Of major concern to our industry is the fact that this widely used material is not recognized per se in the document. As a general recommendation we feel that creation of a separate subsection for precast and prestressed concrete is fully warranted, considering the unique uses of the design aspects of seismic design associated with this material, and in light of the extensive use of the material in seismic zones. A separate subsection under Chapter II (Reinforced Concrete) should be established. This subsection should address the use of lateral load resisting systems and components, as well as proper design of non lateral load resisting components to assure their ability to accommodate movements and distortions of the structure under seismic loading without suffering structural distress or contributing inadvertantly to the stiffness of the lateral force resisting system of the building. The effects of vertical acceleration, as well as lateral forces, on precast and prestressed concrete should be covered. This subsection should be further divided into the following categories:

- Plant cast prestressed concrete
- Post tensioned concrete
- Plant cast precast concrete
- Site cast precast concrete

This would enable us to group in one section requirements for precast and prestressed concrete that are at present scattered about the entire document; also some essential seismic design aspects, such as the ductile design of precast concrete cladding connectors, are omitted entirely. This treatment is analogous to the distinct treatment currently afforded precast and prestressed concrete in UBC-79 and ACI 318-77. An additional proviso should be added in this subsection similar to those apprearing in UBC-79 and ACI-77, to wit:

> "All provisions of this document shall apply to precast and prestressed concrete except as specifically modified herein."

There are also several items in the body of ATC 3-06 that we feel should be modified. These are discussed briefly below:

1. Section 11.2 - Connections of Precast Components. Assigning an arbitrary low value of capacity reduction factor to connections does not adequately assure proper performance under seismic conditions. The connections of precast elements should be able to achieve ductility and at the same time maintain their anchorage integrity within the concrete. This requirement should be modified with a statement that ensures proper performance of the connector to accommodate maximum induced drift movements of the structure.

2. Section 11.2 - Axial Compression. The arbitrary assignment of a low capacity reduction factor for a "pin-ended" column is not warranted when the top and bottom connections are designed to accommodate maxiumum drift movements and increased bending movements induced by the P-Δ effect.

3. Sections 3.3.4 and 3.3.5 - Height Limitation. Once again, we have an arbitrarily assigned value for height which is inconsistent with the evidence of performance of shear wall and braced frame buildings in recent earthquakes (Managua, Romania). Proper building design and location of stiffening elements should be the controlling factor, as is alluded to correctly in Section 3.4.1.

4. Section 11.8.2 - Diaphragm Details and Limitations. No provision is made for "untopped" diaphragms consisting of grouted castellated shear keys and boundary closure pours, or untopped precast elements tied together with shear friction reinforcement as described in Section 2611 (p) of the Uniform Building Code. Recent test information is available substantiating the effectiveness of untopped floor diaphragms in transmitting lateral forces.

5. <u>Section II.8.4 - Boundary Members</u>. Some of these provisions are not warranted in light of current practice and research in the area of large panel precast concrete systems building. Tests by Bertero show that boundary elements assure continued flexural behavior under extreme seismic loading. Becker and Mueller have achieved ductile behavior with coupled walls without boundary elements. Once again, it is the design of the structure that is critical; this cannot be assumed by arbitrarily assigned restrictions such as we see here.

6. <u>Section 7.4.4 - Special Pile Requirements</u>. Concerning the anchorage of piles to pile cap, the use of field placed anchor dowels grouted into sleeves cast in the pile top should also be allowed, as outlined in CAL-TRANS Standards and Specifications, Section 49-1.09.

7. <u>Section 7.5.3(c) and 7.6.1 - Special Pile Requirements and Limitations</u>. The use of prestressed concrete piling should not be precluded in seismic categories C & D; performance requirements should be given for their design. A proposed revision to this section is furnished to the Council under separate cover.

8. Section 8.2.2 - <u>Lateral Design Forces on Nonload Bearing Cladding Panel Elements</u>. The forces assigned are in some cases in excess of current code requirements as outlined in UBC-79 - Table 23-J. At the same time, the connector fastener should be designed for forces well in excess of the values assigned. This section should also emphasize the necessity of the connector body yielding to achieve ductility in lieu of approaching forces which would cause fracture of welds or brittle failure of concrete at the connector embedment location.

Many of these items were first discussed in a letter to the National Bureau of Standards dated 2 June 1978. We have not had a response to this communication. I have included a copy of that letter with these comments. Included with the letter is an extensive bibliography of papers concerning precast and prestressed concrete design and construction, which contain a wealth of substantiating evidence for some of the comments we have made here.

Thank you for your consideration.

Very truly yours,

David A. Sheppard

David A. Sheppard
Prestressed Concrete Institute

R14

sepp firnkas engineering, inc. 251 newbury st., boston, mass. tel. (617) 267-6715

June 2, 1978

United States Department of Commerce
National Bureau of Standards
Washington, D.C. 20234

Attention: Mr. Charles Culver, Disaster Research Coordinator,
Center for Building Technology
Reference: Review of "Tentative Provisions for the developmant of Seismic regulations for buildings".

Dear Mr. Culver,

On behalf of the Prestressed Concrete Institute, I have reviewed the "Tentative Provisions" and would like to comment on the attached sheets.

I have tried to be specific and positive wherever possible and due to the time restraints I have commented only on subjects directly or indirectly related to precast and or prestressed concrete elements or assemblies of these elements. As a member of the PCI Seismic Committee and the MIT Seismic Research Review Board I have had the opportunity to keep abreast of the most recent design and research developments in this particular field, and as a practicing design engineer had occasions to apply Seismic codes to actual design and experience the intracacies of interpretations of codes.

It is in this spirit that the comments should be taken.

May I state in behalf of PCI that we are extremely interested to contribute to the further development and refinement of the "Tentative Provisions".

Yours truly,

sepp firnkas engineering, inc.

Sepp Firnkas

Sepp Firnkas

SF/ezk
enc.

cc: Mr. Daniel Jenny, Prestressed Concrete Institute

sepp firnkas ulrich boehlke chan lin lawrence ogden

firnkas engineering, inc.

June 2nd, 1978

Review and Comments on "Tentative Provisions for the Development of Seismic Regulations for Buildings".

I. General

Precast concrete is referred to once in Chapter 11: "Reinforced Concrete" indicating a capacity reduction factor of $\phi = 0.5$ for connections of precast components, and then in the commentary where it states:
"The experience, both in the field and in the laboratory, which has led to the special proportioning and detailing requirements documented in this Chapter for Categories C and D has been predominantly with monolithic cast-in-place reinforced concrete construction. Therefore, these requirements must be projected with great caution to types of construction which differ in concept or fabrication. Precast reinforced concrete elements may be used as part of the seismic resisting system provided their strengths, proportions, and details can be demonstrated to comply with the requirements stated for Categories C and D construction."

Ch. 7 "Foundation Design requirements" mentions briefly the tie and longitudinal steel requirements in the upper part of PRECAST pile sections and further in sections 7.4.4 (D) & (E), 7.5.3 (C) and in 7.6.1 which eliminates precast prestressed concrete piles from resisting flexure caused by earthquake motions for buildings of performance category D. The commentary to Section 7.6 amplifies this with: "At the present time, there is little or no information available on the ductility capacity of precast-prestressed piles; in fact, the type of reinforcing provided is counter to present concepts of concrete ductility development. Hence, until further data are available, they should not be used in situations where pile bending may be induced by earthquake motions.

A short review of the Bibliography of Ch. 7 and 11 indicates not a single reference to precast and/or prestressed concrete elements, connections or assemblies. Considering the above a review of the "Tentative Provisions" specifically as far as precast and/or prestressed concrete and its numerous applications are concerned is handicapped and can therefore only be general. I will, however, try to introduce as many specific subjects as time will permit.

2. Chapter 3. "Structural Design Requirements". Section 3.3. :
4 types of general framing systems are mentioned - table 3-8 lists 5.
This should be clarified.
Sections 3.3.4 & 3.3.5: The height limitations imposed on pure shear wall or braced frame systems seem arbitrary and should seriously be reviewed in view of the performance of such buildings in recent earthquakes - Managua for example - Ref. (185) and research references (186, 187, 2, 5, 7, 8, 165, 166, 167 and 189).
Section 3.4. : A regularity in plan configuration should be required for Categories C & D. This is a discipline the engineering - ultimately responsible for Safety in E.Q's - should impose on architectural whims.
1. - Re-entrant corners of significant dimensions should be separated by adequate separation joints from the main part of the building to create independently stable buildings.
2. - Torsional moments should be minimized by controlled layout.
3. - Controlled layout can provide significant diaphragm strength.
Section 3.4.2.: Same comments as above apply.

The Architects' preference for an open ground floor or a building on stilts should seriously be discouraged for Categories C & D. Locations of expansion joints should be directed by symmetry. The Earthquakes of Caracas 1967 and Alaska 1964 - Ref. (183, 184) demonstrated clearly the importance of a controlled regular plan and verical layout.

3. Chapter 7. "Foundation Design Requirements".
Section 7.4.4. (D) & (E): It is not obvious at this point why a distinction between precast and precast & prestressed piles should be made. The longitudinal reinforcing is mostly determined by handling and driving stresses. The minimum tie spacing should be equal for both types and specified considering that most of the time the upper portion of the pile is cut off i.e.: the minimum tie spacing after cut off should extend 2' below the bottom of the pile cap.
Section 7.5.3 (A) & (C): It is difficult to see in this connection the difference bewtween a cast-in-place and a precast concrete pile.
Section 7.6.1 : The commentary to this section states the lack of information on ductility of prestressed concrete piles as a reason to eliminate them in Category D to resist flexure caused by earthquake motion. This is obviously an "easy way out" instead of assemblying and evaluating available research and performance data.

Prestressed concrete piles can be compared to prestressed columns which have been researched and used for a long time - Ref. (56, 53, 188 and others) and performed for many years in offshore and marine structures and for bridge foundations in the open sea subject to continuous wave and live load actions. Extensive literature and research has been published in the PCI Journal by FIP and can be obtained from "Raymond International" (the former Raymond Concrete Pile Co.); "Brown & Root Inc.", "The Bayshore Concrete Corp." and European Sources.

Upon studying these data it can be concluded that prestressed concrete piles do have superior flexural qualities in the elastic stage and seem ideal for energy dissipation and damping and it appears that they are preferable to just plain concrete piles, thin steelshell concrete filled piles or piers.

Since the "Provisions" cover extensively all structural systems, the omission of prestressed (posttensioned) concrete mat foundations may suggest to the average reader a negative aspect. The increased use of posttensioned mats especially in difficult soil conditions (expansive clay, fine sands, silts, subsidence areas, etc.) and its ability to bridge areas of weakness has also great advantages in case of ground motions that may disturb the regularity of supporting strength of the subsoil.

Theoretical and practical backup data could be obtained from a similar structural system i.e. from prestressed flat plates and the Posttensioning Institute.

4. Chapter 11.
It is stated in this chapter that precast reinforced concrete components may be used if the resulting construction complies with the requirements of Section 3.6 and this chapter. The comments elaborate further that: "......these requirements must be protected with great caution to types of construction which differ in concept or fabrication. Precast reinforced elements may be used .... provided their strength, proportions and details can be demonstrated to comply with the requirements stated for categories C & D construction". This is fair but sounds like a negative acceptance of precast reinforced concrete. It is accepted that the burden of proof lies with the newcomer, but it should be the obligation of the investigator to avail himself, study and evaluate all published research and experience data on the subject. Further should be recognized that precast reinforced concrete elements may have similarities with cast-in-place concrete, however the force reansfer details of precast concrete elements do not have a counterpart in cast-in-place concrete joints.

The attached references of type 1 - marked with a * are concerned with behavior of precast and/or prestressed elements subject to seismic motions, while the references of type 2 - marked with ** investigate the performance of various types of connections and joints.

Section 11.2
To assign a single even if low ϕ factor to connections does not insure ductility. In practically all connections is steel and concrete involved - two materials with quite different characteristics. Concrete is brittle compared to steel. To achieve ductility of the connection a yield of steel elements at or near the concrete interface must be obtained while the concrete stresses should at this stage remain below rupture.

Recent PCI Special Committee and research work contained in "Mannual for Design of Architectural Precast Concrete" Chapter 2 - Connections resolved the problem by using equal strength design for steel and concrete and separate ϕ factors of $\phi_s = 0.90$ and $\phi_c = 0.65$ respectively to assure ductility. This method allows a fairly uniform design for ductility for complex connection details where sometimes 6 force transfers are involved.

Since precast-prestressed concrete is an established construction method and industry - and will be more so in the future, the "Provisions" should include realistic evaluations of behavior and performance of connections and not dismiss it in general terms.

Section 11.8.2.
The last paragraph about topping is vague and may lead to misinterpretations. It has been proven experimentally and can be substantiated analytically that the shear key connections between individual floor elements (shear-friction concept) can transfer diaphragm forces usually in excess of the design forces.

The effectiveness of a thin topping slab on floor elements seems questionable if the floor slabs are tied together by longitudinal, transverse and peripheral ties according to current design standards - Ref. (155). There are several full scale test results available demonstrating the satisfactory behavior of grouted untopped floor elements subject to "in plane" forces.

Section 11.8.4.
This section should be reviewed in its entirety as to its applicability to precast concrete systems construction. The most recent research conducted at MIT - Ref. (3,4,5,6,7 etc.) and the various Japanese, Russian and Yugoslavian test results should be used to develop guidelines for precast concrete systems.

Boundary members for diaphragms - See Ref. (155).

Table 11-A.
The shear and tension values for bolts given in this table should be checked. Since seismic forces cause most of the time shear and tension an interaction diagram should be given instead of separate values for shear & tension. See Ref. (106).

ATTACHMENT B

Response of the Concrete Industry to ATC 3-06
Presented to
Building Seismic Safety Council
San Francisco, CA
November 8, 1979
by

Mark Fintel, Director, Advanced Engineering Services
Portland Cement Association, Skokie, Illinois

INTRODUCTION

I appreciate this opportunity to present the reponse of the Concrete Industry to ATC-3. First of all, we would like to express our compliments to the Applied Technology Council for preparing a document which, in many respects, advances seismic design concepts a long step forward. New approaches to seismic zoning, to soil-structure interaction, and to period determination, are specific examples of improvements utilizing knowledge gained during recent decades, which will affect the daily practice of earthquake engineering.

On the other hand, many shortcomings contained in present practice, especially as related to special moment frames for reinforced concrete, are continued within ATC-3. These provisions make it practically impossible to construct highrise concrete structures in areas of high seismicity.

Looking back at the history of earthquake codes related to reinforced concrete multistory structures in the United States, we find that in 1959 the SEAOC recommendations restricted the height of concrete buildings to 13 stories or 160 ft.

As a direct response, the concrete industry produced, in 1961, the book entitled "Response of Multistory Reinforced Concrete Buildings to Earthquake Motions" by Blume, Newmark and Corning, which firmly established the fundamentals for earthquake resistance of reinforced concrete multistory structures.

At that time the specified requirement was that concrete structures must show the same ductility as structures constructed of A7 steel; also, the prevalent moment-resisting frame led SEAOC and the concrete industry to develop the Ductile-Moment-Resisting-Space-Frame (DMRSF).

In 1966, the 13-story height limitation was removed and provisions requiring the DMRSF system for concrete buildings above 13 stories were introduced. Since that time, only about half-a-dozen DMRSF structures of reinforced concrete, taller than 160 ft, have been built in California.

It is only in retrospect that we can see our mistake in not pursuing the development of ductility in shear walls in the 60's. At that time we did not clearly foresee that legislating concrete into structural forms suitable and desirable for steel may not work. In concrete we cannot emulate steel; we need to utilize the inherent rigidity and strength of concrete walls for lateral resistance of buildings. Reinforced concrete walls have been penalized by Codes because of their supposed lack of ductility; however, recent experimental and analytical research has clearly shown that walls can be made ductile when properly proportioned and reinforced.

The reason that highrise concrete buildings are not built in California under existing codes is not that reinforced concrete is a material unsuitable for seismic resistance. Rather, it is that we require ductility where it is not needed, thus creating expensive, even unbuildable, structures. This is a direct result of the use of elastic analysis under Code-specified static loads, which cannot give us a proper assessment of ductility requirements. We have not yet adopted the more realistic inelastic analysis techniques for structural response which have been developed in recent years. ATC-3 needs to consider these.

All indications are that ATC-3, in its present form, will not improve the buildability of highrise concrete buildings in seismic areas.

To more clearly assess the effects ATC-3 provisions may have on concrete buldings, let us examine, separately, the three basic components which comprise the design process, and which directly affect safety and economy.

These are:

- loading;
- overall concept (including analysis and design), and
- proportioning.

SEISMIC LOADING

The new seismic maps incorporated into ATC-3 which consider historical records of earthquake occurrences, and also distance from earthquake sources, have created a much more realistic basis for assessment of earthquake forces. Also, such apparent inconsistencies as requiring the same level of seismic forces for Boston as for San Francisco have been eliminated. The new maps offer a much finer gradation of earthquake intensity across the United States, and will make it possible to more effectively employ rational approaches to aseismic design of buildings, thereby reducing expensive overdesign.

OVERALL CONCEPT - Analysis and Design

Taking a bird's-eye view of the state-of-the-art, we see that the introduction of earthquake response spectra more than three decades ago constituted a major step forward in seismic engineering. Also, the introduction of the ductility concept brought a perspective of realism to the design of buildings.

In recent years, the academic profession has developed powerful computer programs which permit inexpensive inelastic response history analyses of buildings subjected to ground motions. Also, the experimentalists have accumulated an extensive body of knowledge about the strength, stiffness and ductility of the individual structural elements and their assemblies when subjected to cyclic reversing loads. However, when we look at the implementation of this vast store of recent knowledge into the practical design of our buildings, we find a wide gap. ATC-3 is continuing to use the same static elastic method of analysis to determine seismic forces and deformations that we used 30 years ago. This is not appropriate for structures which respond inelastically.

The present overall philosophy for seismic design of yielding structures continued in ATC-3 is based on a balance between strength and ductility. While the basic concept is undoubtedly valid, its implementation as currently practiced has shortcomings with respect to strength, and particularly with respect to ductility. The concept is implemented in present codes primarily through K-factors, and also through load and understrength factors, and through detailing requirements for ductility.

Strength. Elastic static analysis cannot adequately determine forces and deformations in inelastic structures. Designing on the basis of elastic analysis may lead to inadvertent shear failures, such as observed in the Banco de America building in the 1973 Managua earthquake. Unfortunately, this analysis is still in the forefront of the ATC-3 approach.

Ductility.* ATC's implementation of the concept of ductility within the design process, which is in accordance with current practice, causes major construction difficulties and unnecessarily increases cost, without

* We are continuing to use the term "ductility" with hesitation, since no other term has been advocated. Whatever term is used, we are talking about energy dissipation associated with damping and yielding in the structure.

improving safety or performance. Originally, the concept was developed on the basis of studies of single-degree-of-freedom systems, and system displacement ductilities of 4 to 6 were utilized for a 1940 El-Centro type earthquake. However, in designing a structure, we deal with the ductilities of individual members, and not with overall system ductility. The relationship between the two may be different for each member in a structure, and changes of structural configuration will result in changes in the individual member ductilities. Therefore, while we are talking about system ductilities of 4 to 6, we may be faced with member rotational ductilities several times larger, depending on the structural configuration, and strength and stiffness relationships. No systematic studies have been carried out to determine the distribution and magnitude of member ductilities within a structure. Consequently, in the present implementation of the concept to assure safety against brittle failures, we must, of necessity, provide maximum ductility in all columns, beams, and their connections, whether needed or not. In reality, from experience in earthquakes, and from inelastic analyses, it is known that ductility is not required in all members of a frame. Unfortunately, this important economic consideration is not included in ATC-3.

Elastic analysis. The major drawback of elastic analysis when applied to inelastic structures is that it does not allow us to determine the amount and distribution of ductility throughout the structure. We hope that the details specified in ATC-3 and other seismic codes will assure the required ductility in all members which may become inelastic. We hope--we do not know for sure.

Other shortcomings resulting from the use of elastic analysis for an inelastic structure are the possibility of inadvertent yielding of columns during very severe earthquakes with its consequent effects on overall structural stability, and also the lack of an active control over the sequence of yielding during seismic response.

Just a few years ago we could not have commented about the unsuitability of elastic analysis, because we had no practical alternative. Today, with the availability of inexpensive inelastic dynamic response history analyses and inelastic static analyses, we do have a practical alternative. Significant progress towards the development of a practical inelastic analysis and design procedure, including design examples, has been accomplished in recent years at PCA. This procedure can be used to design structures which warrant dynamic analysis. More importantly, we can use this procedure to verify the present and other suggested design approaches, to weed out the inadvertant deficiencies which may result from the use of elastic analysis.

Modal Superposition Analysis. The dynamic analysis suggested in ATC-3, using the modal superposition method, is reasonably accurate for elastic structures such as nuclear power plants. However, for structures intended to yield in their earthquake response, it has the same drawbacks as the empirical method, since it relies on elastic static analysis for member forces and deformations. The amount and distribution of ductility cannot be determined, and therefore, we must indiscriminately provide ductility in all members.

Response Modification Factors. The concept of response modification factors introduced in ATC-3 to account for the inelasticity and damping of the various structural systems and materials is conceptually clear, simple and easy to apply. It has the potential to update, refine and improve the previous K-factors, provided the R-factors can be established with at least a reasonable degree of confidence. However, the way the R-factors have been compiled in Table 3B, makes the implementation of the concept very questionable at this time. Using the judgment of a small group of individuals, however knowledgeable, to arbitrarily select the R-factors, without the resource of any published background material, and without appraisal by the profession and by the concrete and steel industries, raises more questions than can be answered.

Determination of a reliable table of R-values and its correlation with the required and available member ductilities must not be done on an arbitrary basis. Ductility of members is greatly dependent upon geometry of the structure, and upon strength and stiffness interrelationships; each change in the internal makeup of a structure causes changes in the amounts and distribution of member ductilities.

To evaluate the suggested arbitrary Response Modification Factors, R, of various individual systems and materials by comparing them with the previous "K" values adopted arbitrarily 40 years ago is like the blind leading the blind. Acceptable "R" values can be derived only with the help of inelastic response studies.

As long as ATC-3 continues to specify elastic analysis in conjunction with the suggested arbitrary R-factors ranging from 1-1/4 to 8, any further sophistication of the seismic loads is of questionable merit.

No criticism is constructive unless accompanied by suggestions for improvement, so we would like to make the following specific proposal for the determination of R-values:

The studies to determine R and C_d values must be carried out for the various structural systems and materials of Table 3B. For each system type, structures with varying periods within the practical range must be considered. In the analysis of each structure, one must use a set of several ground acceleration time histories corresponding to the target response spectra. In these analyses, the strength levels in the structures should be adjusted so that the ratio of the base shear calculated from an inelastic response history analysis to the base shear from the undamped elastic response history analysis under the same ground motion will equal 1/R. The inelastic response history analyses would yield required member ductilities corresponding to the prescribed R factors. If these required ductilities are attainable with the specified detailing, then the prescribed R-factors are realistic; otherwise, they need revision.

We recognize that the total effort required is very extensive. However, it must be undertaken and systematically carried out, if the proposed design provisions are to be based on a solid foundation.

It is hoped that the dynamic inelastic response history studies proposed above for both concrete and steel structures will also lead to a relaxation of the ATC-3 requirement that ductility be provided throughout an entire structure, while it may actually be needed only in certain specific locations. For instance, in shear wall-frame structures, it is unlikely that ductility provided in most columns can ever be utilized. A recognition of this fact would make concrete structures much more practicable in seismic regions.

Height Limitations. The height limitations for the various framing systems, as given in Section 3.3 seem questionable. The best performer in reinforced concrete, the shear wall-frame interactive system, has justifiably been assigned a high R-factor. The shear wall-frame, however, is limited to 240 ft, while the special moment frame, which in reality becomes unbuildable at about 15 stories, is the only concrete system allowed above 240 ft. We do not believe that the reasons and circumstances which prevailed in the early sixties and led to such height limitations (primarily a lack of knowledge) are still valid today.

PROPORTIONING

While most provisions of Chapter 11, Reinforced Concrete, are based on the present state-of-the art, there are a number of specific details inconsistent with available research results and current building code requirements. For example:

ASTM designation A615 Grade 60 reinforcement is not permitted, even though it is currently in wide use. Rather, only A706 reinforcement is permitted, a steel that is not readily available and one for which a premium price has to be paid. This economic penalty is not justified. Also, despite the requirement of A706 reinforcement, load factors on joint details and shear walls are based on tests using A615 Grade 60 bars.

Similarly, all lightweight aggregate concrete is penalized by unrealistic restrictions on design strengths. Research data do not justify this limitation.

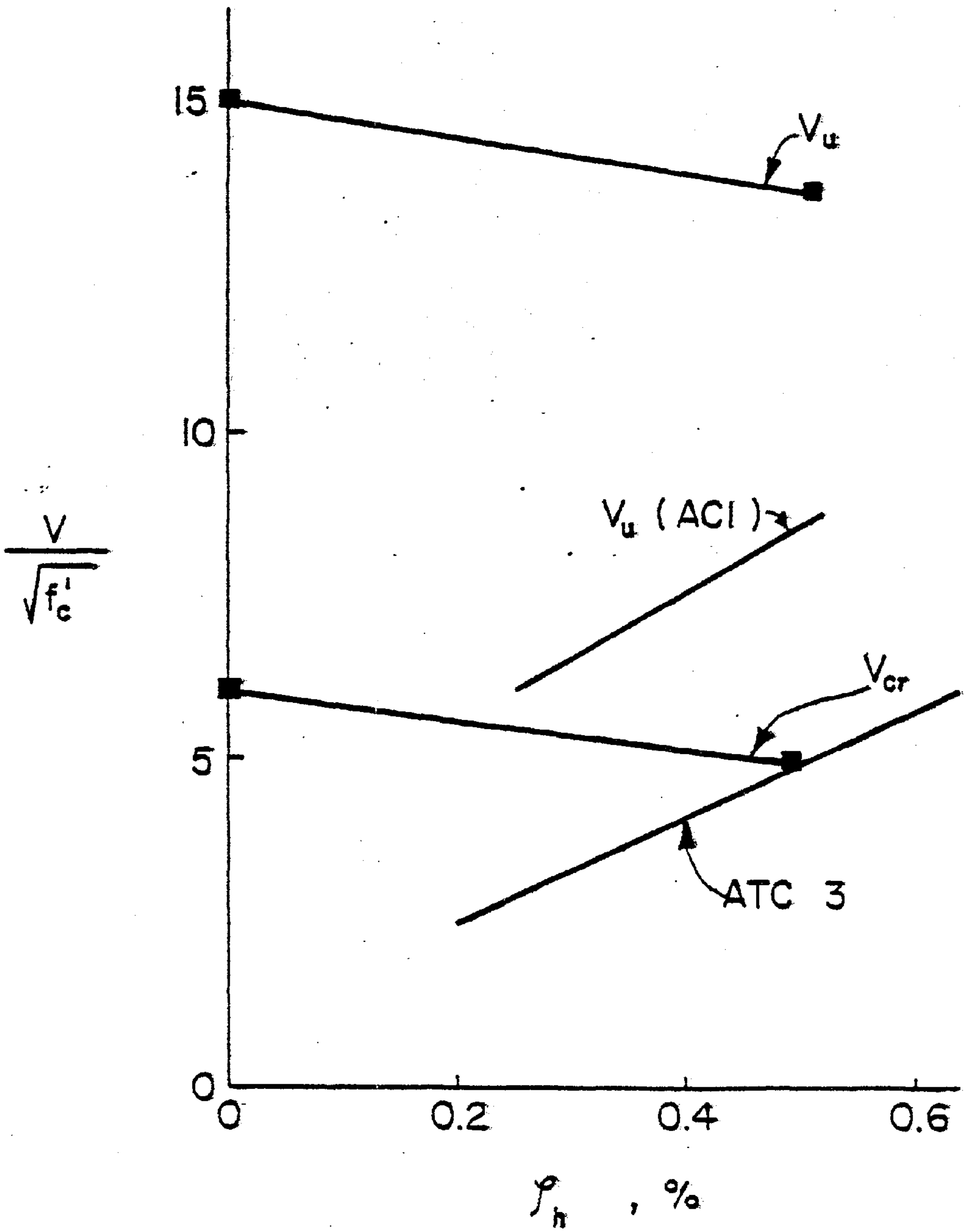
One of the most restrictive suggestions in Chapter 11 is Equation 11-5. The enclosed graph shows the proposed ATC-3 design requirement compared with results of tests on short walls. On the ordinate, shear strength is plotted, and on the abscissa, the amount of horizontal reinforcement. The top line indicates measured strength, the line marked v_u (ACI) is strength permitted by ACI 318-77, the line marked v_{cr} is the observed cracking strength and the bottom line is the strength proposed by ATC-3. As can be seen, at ultimate load the walls would have a load factor of 1.5 or more against cracking. The load factor on strength could be as high as 14.

CONCLUSIONS

There can be two reasons for code changes: either the old provision has been shown in the field to be unsafe or too conservative, or a new provision is known to produce better structures. In the case of the R-factors, neither of these is clearly the case. It would seem prudent to continue with the present K-values until we are ready to use a more rational approach to structural response with an accuracy which is not only implied, but is actual.

ATC-3 has continued to impose on multistory concrete buildings requirements which unnecessarily increase their cost. To provide ductility in places where it can never be utilized makes our buildings more expensive, without adding an iota to their safety. We need to incorporate inelastic behavior into our design process so that we can incorporate ductility details only where they can be utilized. This item merits urgent consideration by the Applied Technology Council.

SHEAR STRENGTH



Minutes of Technical Committee 4

Review and Refinement of ATC 3-06

February 21, 1980

San Francisco, CA

9:00 AM PST Meeting is called to order by Chairman Edward Cohen. Those present are:

Name	Representative Of
Edward Cohen (chairman)	American Concrete Institute
James Prendergast	Interagency Committee on Seismic Safety in Construction
Joseph Manning	Concrete Reinforcing Steel Institute
James Lefter	Building Seismic Safety Council
Vitelmo V. Bertero	Applied Technology Council
Neil M. Hawkins	Post-Tensioning Institute
Mark Fintel	Portland Cement Association
David A. Sheppard	Prestressed Concrete Institute
Richard Marshall	National Bureau of Standards
Kyle Woodward	National Bureau of Standards
Robert Park	Guest

Discussion began with consideration of the recommendation made by Mr. Cohen (see letter dated 2/11/80) to adopt the draft revision of ACI 318-"Building Code Requirements for Reinforced Concrete," including Appendix A, as the reference in Sec. 11.1. The draft version of Appendix A is expected to go before the full ACI 318 Committee at the ACI Spring Convention, March 3-8, 1980.

Mr. Fintel suggested that trial designs could be based on the draft provisions of Appendix A because few substantive changes to the Appendix A provisions are anticipated prior to its final adoption.

Mr. Lefter suggested that the committee must make a decision on the use of Appendix A before proceeding with future committee work.

Mr. Bertero raised strong objections to the use of the draft Appendix A provisions. He suggested that it would be improper to adopt them because they have not been officially adopted by ACI 318. Mr. Bertero also suggested that certain incompatibilities exist between the ATC 3-06 document and the Appendix A provisions, especially with regard to the way in which loads are defined. He recommended that the present provisions in Chapter 11 of ATC 3-06 be improved rather than adopt the draft Appendix A provisions.

Mr. Sheppard pointed out that there is at present no mechanism for regularly updating the ATC provisions. By adopting Appendix A, any changes resulting from the ACI review process would automatically be incorporated into the ATC provision. In addition, since ACI is a consensus code and has national representation, the provisions would be more widely accepted.

Mr. Fintel suggested that since ATC 3-06 is not a legal document as yet, there would be no problem in using the draft Appendix A.

Mr. Bertero expressed doubts that the Joint Committee on Review and Refinement would accept the legality of using the draft Appendix A.

Mr. Marshall explained that the committee's purpose is to develop a Chapter 11 which can be used in conducting trial designs. The provisions of Chapter 11 will very likely be revised on the basis of trial design results. The legality of the draft is not at issue. Only the technical aspects of the design recommendations need to be considered.

Mr. Fintel suggested that a section in Chapter 11 be included to serve as the necessary link between the Appendix A provisions and the remainder of the ATC provisions. Any incompatibilities could be resolved in this section.

A general discussion relating to the adoption of the draft Appendix A provisions led to the following recommendations:

a) The committee recommended that the Coordinating Committee be asked to consider the proposal to adopt the draft Appendix A.

b) The committee recommended that the proposed use of the draft Appendix A provisions be discussed with ACI Committee 318 at the ACI Spring Convention.

c) Mr. Lefter will suggest to ACI 318 subcommittee 10 (Appendix A) that reference be made in Appendix A to the ATC zones for earthquake loading.

d) Mr. Neville, ACI 318 Secretary, will be asked to write the transition section in Chapter 11.

e) The committee recommended that the draft Appendix A provisions, with the necessary interface, be adopted as Chapter 11.

Mr. Hawkins suggested, and it was generally agreed, that Appendix A be modified to recognize precast and prestressed construction. He suggested that for category B buildings the current Appendix A provisions are too restrictive.

<u>Break</u>

The committee began discussing Mr. Bertero's comments on the proposed revisions submitted by Mr. Sheppard, Mr. Forell (SEAONC representative to Technical Committee 2), Mr. Fintel, and Mr. Manning.

Mr. Sheppard's proposals were discussed first. (Mr. Sheppard's proposals are contained in the letter dated 12/21/79. Mr. Bertero's replys are in the letter dated 1/31/80 with the cover letter from Mr. Marshall dated 2/7/80.)

1.1 Section 11.2

Mr. Sheppard and Mr. Hawkins both suggest that specific mention be made of precast and/or prestressed concrete construction. Mr. Hawkins stated that the current ACI 318 Building Code deals only with site cast monolithic reinforced concrete construction.

Mr. Bertero suggested that a completely new section or chapter be developed to cover precast and/or prestressed construction rather than adding something in Chapter 11.

It was agreed that the committee should recommend to ACI 318 that provisions for precast and/or prestressed construction be developed, including seismic considerations.

Mr. Bertero noted that the required detailing provisions would vary for each of the different types of precast and/or prestressed construction.

Mr. Hawkins suggested that detailing provisions for certain specific types of construction could be developed immediately. Mr. Hawkins and Mr. Sheppard agreed to draft a section describing the construction type and detailing provisions. The draft is to be submitted to the committee within one month. A paragraph in section 1.3.1 will be added to refer the reader to the appropriate section in Chapter 11.

The wording of section 11.2 is to be changed to include the phrase "precast and/or prestressed" instead of "precast."

1.2 Section 11.2

Mr. Bertero's comment that the proposed revision should not be accepted was adopted for the reasons stated by Mr. Bertero.

Adjourned for Lunch

1.3 Section 11.8

Mr. Sheppard agreed to Mr. Bertero's modification.

Mr. Fintel questioned the use of only the topping as the load resisting mechanism. He considered it to be overly conservative and restrictive.

Mr. Hawkins suggested that Mr. Bertero's modification be altered by deleting "...like those expected for seismic performance categories C and D." and inserting "For buildings in performance categories C and D" at the beginning of the last sentence.

Mr. Bertero's modification with Mr. Hawkin's change was adopted by the committee.

1.4 Section 11.8

Mr. Bertero's rejection of the proposed revision was accepted by the committee for the reasons given by Mr. Bertero.

1.5 Section 11.9 to 11.12

Not discussed again because the idea of separate provisions for precast and/or prestressed construction had already been discussed.

1.6 Section 11.2

Mr. Bertero's rejection of the proposed revision was accepted by the committee because it was generally agreed that providing confining steel is good design practice.

1.7 Section 3.3.4

1.8 Section 3.3.4

There was a general discussion of the meaning of the height limitations and for what types of systems they applied.

Mr. Cohen suggested that paragraph 3 on page 339 (commentary) limited dual systems to heights less than 240 feet.

Mr. Fintel suggested that this was not what was actually intended. He stated that the 240 foot limitation applied to single wall systems only.

Both Mr. Bertero and Mr. Manning suggested that the committee recommend a change in wording of the relevant sections to establish coupled shear wall systems as a separate definition. The committee agreed that the section needed clarification and recommended that Committee 2 study it.

1.9 Section 3.3.5

Mr. Bertero stated that the limitations were to emphasize the need for not only survivability, but functionality after an earthquake for performance Category D buildings.

1.10 Sections 7.4.4, 7.5.3, and 7.6.1

Mr. Bertero agreed with the general concept of using prestressed piles, but pointed out that there are a number of variables still undefined in soil-structure interaction.

Mr. Sheppard repeated that his proposed revision was based on published results and methods. Mr. Sheppard presented the information to Mr. Marshall to be copied and distributed to the members of the committee.

Mr. Fintel proposed that Mr. Sheppard go before the Foundation Committee (3) to present the views of Committee 4.

Mr. Bertero suggested that prestressed piles should not be excluded because their performance demonstrated adequate deformational capacity if properly detailed.

Mr. Cohen proposed that Committee 4 present the data on prestressed pile performance to Committee 3 and recommend that the restriction against using prestressed piles in Category D (Section 7.6.1) be deleted. Mr. Cohen proposed that Committee 4 develop provisions to be included in Chapter 7 that would specify the necessary detailing requirements for prestressed piles used in performance Category D. Mr. Sheppard agreed to prepare draft provisions for the detailing requirements and submit them to the committee within one month.

The committee agreed with Mr. Cohen's proposals.

Mr. Cohen suggested that the provisions of section 7.4.4 be moved to the appropriate materials-specific chapters.

Mr. Hawkins objected because certain pile types have no corresponding materials-specific chapters.

Committee 4 agreed to suggest to Committee 3 that the wording in section 7.4.4 be altered to allow dowels embedded in pile caps as well as dowels embedded in the pile.

1.11 Sections 8.2.2 and 8.2.3

Committee 4 suggests to Committee 8 that Mr. Sheppard's proposed revisions be reviewed.

Mr. Forell's proposed revisions were discussed.

2.1 Sections 11.8.1, 11.8.2, and 11.8.4

Mr. Bertero's suggested modifications were discussed and adopted.

Mr. Fintel raised questions as to how designers could compute the axial stresses in the diaphragms to know whether or not the provisions of Section 11.8.2 proposed by Mr. Bertero apply. The question was unresolved.

2.2 Section 11.8.4

The committee was unclear as to what clarification was required. Mr. Fintel agreed to request more specific information from Mr. Forell.

<u>Break</u>

Mr. Fintel's proposed revisions were discussed.

3.1 Table 1-B

Mr. Bertero suggested that Tables 1-A and 1-B may unduly penalize reinforced concrete.

Mr. Cohen proposed that in Chapter 11 the level of required detailing could be related to the performance category and seismic area.

Mr. Hawkins expressed concern that the transitions between zones on the seismic map created problems which need additional study.

Mr. Bertero emphasized that the provisions must be directed to the average professional engineer. This requires that detailing provisions account for badly configured systems in seismic areas.

The committee proposes a modified Table 1-B on the basis of Mr. Fintel's revision. The following seismicity indices were suggested:

Map Area Number	Seismicity Index
7	4
6	4
5	3
4	2
3	2
2	1
1	1

3.2 Section 3.3.4

The discussion of height limitations was presented in previous comments.

3.3 Section 3.9

Mr. Bertero expressed concern about emphasizing the use of a unique structural system.

The committee agreed that some specific mention of alternate procedures is desirable. The committee recommends to Committee 2 that the phrase "or on approved alternate procedures." be added to the sentence beginning "The internal forces..." in section 3.1. The commentary should include a reference to the paper by Mr. Fintel (to be written by him) outlining the method developed at the Portland Cement Association.

3.4 Table 3-B

Mr. Bertero suggested that the response analyses of the trial designs could provide valuable information to develop better estimates of R values.

<u>Adjourned for Dinner</u>

The proposed revisions of Mr. Manning were discussed (refer to letter dated 1/31/80 from Mr. Manning to Technical Committee 4 and Mr. Bertero's comments dated 2/11/80).

4.1 Section 11.1

The committee agreed to adopt Mr. Bertero's modification.

4.2 Section 11.2

The committee agreed to adopt Mr. Manning's proposed revision.

4.3 Section 11.5.1

Mr. Bertero pointed out that experience shows that welding Grade 60 reinforcing steel creates problems and requires greater care.

Mr. Manning stated that ASTM A-706 reinforcing steel is difficult to obtain and is gradually being phased out of the market.

Mr. Cohen proposed that the committee eliminate references to ASTM A-706 steel and call for ASTM A-615 steel with the proviso that special welding techniques must be used.

The resulting discussion led to the following proposed change in section 1.6.3.A Add the sentences:

"ASTM A-615 reinforcing steel may be used in place of ASTM A-706 reinforcing steel where no welding of the reinforcing steel is required. ASTM A-615 reinforcing steel may be used in place of ASTM A-706 reinforcing steel and welded only if the provisions of Ref. 11.1, section 3.5 are satisfied."

4.4 Section 11.8.1

The proposed revision was rejected for the reasons given by Mr. Bertero. Mr. Manning agreed with the comments of Mr. Bertero.

Discussion continued on the other proposed revisions submitted by Mr. Manning which were not commented on by Mr. Bertero.

Table 1-B, Seismicity Index

Discussion had already taken place during the review of Mr. Fintel's proposed revisions. The table proposed by Mr. Manning was adopted by the committee and the committee recommends it to committee 2.

Section 3.3.5 Height Limitations

Previously discussed.

Section 4.2.2 Period Determination

Mr. Bertero stated that equations for period reflected the observed trends that r/c structures were generally stiffer than steel structures.

Mr. Manning felt that the current formula for period determination unduly penalized r/c construction. He suggested that the formula be modified on the basis of results from response analyses of the trial designs.

Mr. Bertero felt that the simple formula was adequate and complicated formulas were not justified on the basis of available data.

The committee asked Mr. Manning to review the data on which the period determination formula was based to ascertain its validity.

Committee ended technical discussion.

Next meeting of Technical Committee 4 was discussed. April 14 at the Portland Cement Association was tentatively agreed to.

Meeting adjourned.

Respectfully submitted by
Kyle Woodward
Richard D. Marshall

Minutes of Technical Committee 4

Review and Refinement of ATC 3-06

Meeting at Portland Cement Association

Skokie, Illinois

April 14, 1980

9:00 AM CST Meeting called to order by Chairman Edward Cohen. The following individuals were present:

Name	Representative Of
Edward Cohen (Chairman)	American Concrete Institute
Mark Fintel	Portland Cement Association
Joseph Manning	Concrete Reinforcing Steel Institute
Neil Hawkins	Post-Tensioning Institute
David A. Sheppard	Prestressed Concrete Institute
Loring A. Wyllie, Jr.	Structural Engineers Association of California
James Prendergast (delayed by weather)	Interagency Committee on Seismic Safety in Construction
Non-voting Members	
Vitelmo V. Bertero	Applied Technology Council
James Lefter	Building Seismic Safety Council
Richard Marshall	National Bureau of Standards
Kyle Woodward	National Bureau of Standards
Alternates, Guests and Observers	
Gerald R. Neville	Portland Cement Association
S. K. Ghosh	Portland Cement Association
Daniel Jenny	Prestressed Concrete Institute (Alternate)
Edward O. Pfrang	National Bureau of Standards

The first order of business was the issue of adopting ACI 318-"Building Code Requirements for Reinforced Concrete," including the draft revision of Appendix A, as the reference cited in Sec. 11.1. Mr. Cohen asked Mr. Neville for a status report on the development of Appendix A and changes to Chapter 11, ATC 3-06, that would be required if the provisions of Appendix A were to be adopted as the basis for trial designs.

Mr. Neville referred to the ACI 318-Appendix A draft dated March 19, 1980 and to a revised Chapter 11 (with commentary) which he had prepared and distributed to Technical Committee 4 on March 28. Mr. Neville summarized the changes reflected in his version of Chapter 11 (hereinafter referred to as the "Neville draft") and pointed out that while changes to Appendix A could be expected prior to its adoption by ACI Committee 318, the March 19 draft (hereinafter referred to as "Appendix A") was being proposed as the basis for trial designs.

Mr. Cohen asked the members of Technical Committee 4 for comments.

Mr. Hawkins stated that his review of the Neville draft and Appendix A identified some 15 items that must be revised to ensure consistency with ATC 3-06. Mr. Bertero stated that in his opinion, ATC 3-06 has certain weaknesses in the areas of loads, the treatment of Category B structures, and the treatment of shear. And while he feels that the wording of Appendix A in certain cases represents some improvement over ATC 3-06, Appendix A does not represent any improvement in technical content. Specifically, it does not represent any improvement in the areas just identified.

In response to a question from Mr. Bertero, Mr. Cohen stated that the reasons for considering Appendix A are that a mechanism would be available for its future updating and that ACI Committee 318 is an ongoing activity with wide industry input. He also stated that the intent of Technical Committee 4 regarding Appendix A had been made clear to Subcommittee 10 of ACI 318 and that he was not aware of any reluctance on the part of Subcommittee 10 to see it referenced in ATC 3-06.

Mr. Pfrang noted that the possibility of Technical Committee 4 recommending the adoption of Appendix A was made clear to Committee 318 at the ACI convention in Las Vegas, Nevada. No formal approval for such action was granted by Committee 318 because such approval was not specifically requested. However, he stated his belief that there would be no objection on the part of Committee 318 to such an action.

General discussion followed on what might be done with the work already accomplished by Technical Committee 4 in improving Chapter 11. Mr. Neville stated that these revisions could be channeled to Committee 318 in the form of recommendations for the improvement of Appendix A.

Mr. Hawkins inquired about the treatment of precast concrete in Appendix A. Mr. Neville offered that precast concrete was covered in the Neville draft and Appendix A. Mr. Hawkins and Mr. Sheppard noted that this coverage was not to the same degree that Technical Committee 4 had in mind when it set out to improve and refine the provisions of Chapter 11.

Discussion followed concerning the possibility of assessing the impact of various options (improved Chapter 11 or the Neville draft and Appendix A) during the trial designs. Mr. Pfrang stated that it would probably be too expensive to pursue various optional provisions during the trial designs. Mr. Fintel concurred in this and offered the opinion that the trial designs will probably be more directly affected by the overall design philosophy presented in Chapters 3, 4 and 5 of ATC 3-06.

In the discussion that followed, Mr. Hawkins pointed out that the bond provisions contained in Chapter 11 and Appendix A are significantly different, and that the provisions recommended by ACI Committee 408 (Bond and Development of Reinforcement) were not developed with cyclic loading in mind. Mr. Hawkins suggested that member dimensions would be affected by the differences in bond provisions contained in ATC 3-06 and in Appendix A. Mr. Wyllie noted that Subcommittee 10 was still waiting for Subcommittee 3 to solve the hook problem.

Mr. Sheppard inquired as to the mechanism that would be used to update ATC 3-06. Mr. Pfrang stated that ATC 3-06 was viewed by the BSSC as a resource document, not a code or standard, and that if codes do reference the document in the future, updating should be carried out by a consensus process.

Mr. Bertero stated his opinion that the Committee should move ahead with the refined Chapter 11 and update this chapter after Appendix A has been formally adopted by Committee 318.

The motion was made by Mr. Manning to alter Ref. 11.1 in Sec. 11.1 of Chapter 11 to read as follows:

> Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute. (ANSI/ACI 318-77, including draft Appendix A dated March 19, 1980 — Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions)

The motion was seconded and Mr. Cohen called for discussion. Mr. Neville stated his belief that what is really needed is a vote on his draft of Chapter 11. Mr. Bertero reiterated his objections to adopting Appendix A in lieu of the provisions of the current Chapter 11 with revisions. Mr. Hawkins summarized his list of items that would have to be addressed if the Neville draft of Chapter 11 and Appendix A were to replace the current Chapter 11. Mr. Neville stated that he would be available to make any changes required to mesh the provisions of Appendix A with ATC 3-06. Mr. Bertero stated that he would not have the time to conduct a critical review of Appendix A as he had done to date in the case of proposed revisions to Chapter 11.

Mr. Fintel offered an amendment to Mr. Manning's motion, the amendment being as follows:

> To adopt the draft version of Chapter 11 prepared by Mr. Neville and distributed to the Committee as an attachment to letter to Mr. Marshall dated March 28, 1980.

The motion to amend was seconded and Mr. Cohen called for discussion.

Mr. Sheppard asked for a recess prior to voting on Mr. Fintel's motion.

After a brief recess the Committee voted by secret ballot. The motion to amend was defeated with one vote "yes", four votes "no" and one abstention. Note: Mr. Prendergast did not attend the morning session, having been delayed by bad weather.

Mr. Cohen then asked if there were any questions concerning Mr. Manning's motion.

Mr. Wyllie asked that the original motion be read back for clarification.

The motion was read back by the Secretary and was voted on by secret ballot. The motion carried five votes "yes" to one vote "no".

The next item of business was review of the minutes for the previous meeting of Technical Committee 4 held at San Francisco, California on February 21, 1980.

Mr. Cohen pointed out that most of the issues raised by Mr. Bertero with regard to the minutes (See Mr. Bertero's letter to Mr. Marshall dated March 21, 1980) were now moot in view of the vote on Mr. Manning's motion. Mr. Bertero agreed, but pointed out that the third paragraph under Item 1.8 (page 4 of the minutes) should read as follows: "Mr. Bertero suggested that this was....."

Mr. Sheppard stated that the third paragraph under Item 1.1 (page 3 of the minutes) should read as follows: "...prestressed construction be developed for incorporation into Appendix A." (See Item 1 of Mr. Sheppard's letter to Mr. Marshall dated March 18, 1980)

Discussion followed concerning the development of appropriate R and C_d values for precast/prestressed construction. Mr. Cohen stated that it would be the policy of Technical Committee 4 to pass along all improvements and developments to Subcommittee 10 of ACI Committee 318. Mr. Bertero suggested that a subcommittee could be established to look into R and C_d values appropriate for precast/prestressed construction, but that such a task could not be undertaken by Technical Committee 4 with its limited membership and tight schedule. Mr. Pfrang suggested that Technical Committee 4 could decide to remain active during the trial design period and add members or designate individuals to work through the Committee in developing recommendations for precast/prestressed construction. Mr. Cohen suggested that an ACI-PCA-PCI task force could be organized to develop recommendations for consideration by Technical Committee 4.

Mr. Manning pointed out a misstatement in the minutes, page 7, second paragraph under the heading 4.3 Section 11.5.1. The phrase "and is gradually... the market" was stricken from the minutes.

The conclusion of the discussion regarding the minutes was to accept Mr. Bertero's correction, Mr. Manning's correction, and Item 1 of Mr. Sheppard's letter of March 18, 1980. With these corrections, the minutes were approved.

<u>Adjourned for Lunch</u>

Mr. Cohen reconvened the meeting and asked that the ballot of March 27 be handed in. Those members who had not completed the ballot were requested to return it to the Secretary by April 18.

Mr. Fintel reported on the actions taken by Technical Committee 2 at their meeting of April 2 & 3, 1980, Des Plaines, Illinois. In summary, Technical Committee 2

- rejected by a vote of 6 to 2 all recommendations for changes in seismicity indices in Table 1-B.

- did not vote on recommendation to alter Section 3.3.4 so as to define the coupled shear wall system as a separate category to emphasize that it is a dual system, but did recommend that the BSSC establish a task group to address coupled shear walls.

- approved the addition of wording in Section 3.1 to allow alternate methods of analysis.

- added a section in the commentary describing possibilities and limitations of inelastic analyses.

Attention next turned to the refinements and improvements of Chapter 11 remaining to be addressed by Technical Committee 4. Mr. Hawkins pointed out that Sec. A.2.5.1 of Appendix A should replace the last paragraph of Section 11.5.1. Mr. Manning proposed as new ballot items: (1) that the last paragraph of Section 11.5.1 be replaced by Sec. A.2.5.1 of Appendix A, and (2) that the proposed wording of Item B.2 of the March 27 ballot be replaced by the following:

"Certified mill tests may be accepted for ASTM A-706 and, where no welding is required, for ASTM A-615 reinforcing steel."

Mr. Manning's proposal was put in the form of a motion which carried.

The Technical Committee next considered the issue of seismic provisions for flat plate, flat slab and waffle slab structures as presented in Mr. Hawkins' letter of February 26, 1980. After considerable discussion on the intentions of the ATC 3-06 provisions relating to flat plate construction, Mr. Wyllie offered a motion to consider as a ballot item the changes to Sec. 11.6.1 proposed by Mr. Hawkins and identified as Items 5.1 A-D in Mr. Bertero's evaluation dated April 7, 1980, with Item 5.1 D modified as indicated on page 2 of Mr. Bertero's evaluation. The motion was seconded and adopted.

The following proposals by Mr. Hawkins and identified in Mr. Bertero's evaluation of April 7, 1980 were put in the form of motions and adopted by the Technical Committee as ballot items.

Item 5.2 - As proposed by Mr. Hawkins with modification proposed by Mr. Bertero.

Item 5.3 - As proposed by Mr. Hawkins.

Item 5.5 - As proposed by Mr. Hawkins.

Item 5.6 — Modify current Section 3.7.12 by deleting last sentence.

Mr. Fintel proposed a change in Item A.6 of the March 27 ballot; that the phrase "ductility and stable hysteretic behavior" be replaced by "sufficient energy dissipation capacity." The proposed change was put in the form of a motion, was seconded, and carried. The revised ballot item A.6 will be presented as a new ballot item on the next committee ballot.

The Technical Committee next considered requirements for prestressed concrete piling. Mr. Lefter questioned whether the data on prestressed concrete pile performance referred to by Mr. Sheppard at the meeting of February 21 (San Francisco) was based on cyclic loading. Mr. Hawkins agreed with Mr. Sheppard that cyclic loads were used. Additional information on this issue is to be provided by Mr. Sheppard.

Mr. Sheppard offered a motion that a new Section 7.5.3(E) be placed on the ballot. Wording of Section 7.5.3(E) is to be as indicated in Mr. Sheppard's letter to the Technical Committee (dated March 25, 1980) with the following changes:

Page 1:

- Title to read: PRECAST-PRESTRESSED PILES
- Formula for s_{sp}: change $7\,d_{sp}$ to $7\,d_b$

Page 2:

- Change d_{sp} to d_b = diameter of longitudinal reinforcing or strands.
- Add the phrase "and not less than that given in Section 11.7.2(C)." to the definition of ρ_s in the new Section 7.5.3(E).

The motion was seconded and adopted.

Mr. Sheppard proposed the current Section 7.6.1 be replaced by the wording proposed in his letter to the Technical Committee dated December 21, 1979. Mr. Cohen expressed reservations about requiring a dynamic analysis for steel H piles, but proposed that this issue be placed on the ballot with the understanding that Mr. Sheppard prepare a justification for elimination of the Category D restriction. Mr. Bertero suggested that the major objection to precast piles is the connection to the pile cap. Mr. Wyllie offered to look into the basis for the exclusion of precast-prestressed piles in Section 7.6.1.

Mr. Sheppard made a motion that, as a ballot item, Section 7.6 be deleted and that Section 7.5 be revised to include seismic performance Categories C and D. The motion was seconded and approved.

Mr. Sheppard next proposed as a ballot item the addition of a new section in Chapter 11 to be designated as Section 11.9.2 and as presented on page 2 of his letter to the Technical Committee dated April 8, 1980. After considerable discussion, the following wording was proposed:

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.

The proposal was put in the form of a motion which was seconded and carried.

Mr. Sheppard next proposed for ballot items the new sections 11.9 and 11.9.1 as presented in his letter of April 8.

The following wording was agreed upon after much discussion and was presented as a motion, was seconded and carried.

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.

Mr. Cohen requested that Mr. Sheppard prepare appropriate material for the commentary on Sections 11.9.1 and 11.9.2, and that this material be included on the next ballot. This was put in the form of a motion, was seconded and carried.

Mr. Cohen asked if there were any additional items to be considered.

Mr. Fintel expressed his view that additional work is needed to develop adequate provisions for coupled shear walls. Mr. Bertero agreed and suggested that Technical Committee 4 could recommend to ACI that a committee be appointed to study the issues relating to coupled shear walls. Mr. Cohen suggested that this and other items could be identified and that the Technical Committee could make recommendations to both ACI Committee 318 and BSSC for further study.

Mr. Hawkins noted points of confusion in the commentary on coupled shear walls (page 339, and Figures 3C on page 356). In the interest of improving the commentary on coupled shear walls, a motion was made and seconded that Mr. Fintel develop a new commentary on coupled shear walls, this new commentary to be included as an item on the next ballot.

There being no additional business, the meeting adjourned at 4:10 pm.

Respectfully submitted by

R. D. Marshall

Kyle Woodward

R. D. Marshall
Kyle Woodward

UNITED STATES DEPARTMENT OF COMMERCE
National Bureau of Standards
Washington, D.C. 20234

Minutes of Technical Committee 4
Review and Refinement of ATC 3-06
Meeting at National Bureau of Standards
June 4, 1980

9:00 AM EDT Meeting called to order by Chairman Edward Cohen. The following individuals were present:

Name	Representative of
Edward Cohen (Chairman)	American Concrete Institute
Mark Fintel	Portland Cement Association
Neil Hawkins	Post-Tensioning Institute
Eugene Holland	American Society of Civil Engineers
James Prendergast	Interagency Committee on Seismic Safety in Construction
Loring A. Wyllie, Jr.	Structural Engineers Association of California
Danniel Jenny (Alternate for David Sheppard)	Prestressed Concrete Institute
William Wagner, Jr. (Designated representative for Joseph Manning)	Concrete Reinforcing Steel Institute
Nonvoting Members	
James Lefter	Building Seismic Safety Council
Richard Marshall	National Bureau of Standards
Kyle Woodward	National Bureau of Standards
Guests and Observers	
Gerald Neville	Portland Cement Association

The first order of business was the adoption of an agenda for the Fourth Meeting. This being the last meeting of Technical Committee 4, the Chairman set out the following ground rules prior to consideration of the agenda:

- All business of the committee must be completed before adjournment. There will be no pending actions after adjournment.

- Actions requiring letter ballot approval will be done by signed written vote during the meeting. The votes will be recorded in the minutes for approval by the committee.

- The resolution of reservations from the letter ballots of March 27, 1980 and May 5, 1980 will be completed before the introduction of new business.

It was moved and seconded that the agenda (see Attachment A) be adopted. The motion carried by unanimous vote.

The next item of business was review of the minutes of the Third Meeting of Technical Committee 4 held at Skokie, Illinois on April 14, 1980. No corrections were offered and the minutes were approved as distributed to the committee on April 30, 1980.

The committee next considered the letter ballot of March 27, 1980. Mr. Woodward presented a brief summary of the balloting and noted that of eight voting members, three did not return their ballot. He also pointed out that ballot items A1, A6 and B2 had been superceded by the letter ballot of May 5, 1980. Prior to discussion and final action by the committee, each ballot item was summarized along with the vote tabulation and the reservations and/or reasons for negative votes. In the following notes, results of the March 27 ballot are summarized in parentheses for each ballot item (Y = yes, YWR = yes with reservations, N = no and A = abstain).

A1. This item was superceded by the letter ballot of May 5, 1980.

A2 (Y = 3, YWR = 1, N = 1). Discussion centered on the intent of the proposed change. It was agreed that the proposed change would not circumvent ATC 3-06 requirements for R and C_d values and that ductility requirements would still apply for resistance to lateral loads produced by seismic events. Reservations and objections were withdrawn and the ballot item was unanimously adopted.

A3 (Y = 3, YWR = 2). The reservations concerned the lack of ties at midheight of columns that would result from the proposed change and the corresponding changes in ϕ factors for column design. Following discussion of the issue, the committee voted to adopt the change. Mr. Wyllie requested that the record show his reservation with regard to the proposed change.

A4 (Y = 3, YWR = 1, N = 1). The need for boundary members in the case of tensile axial forces was questioned, as was the basis for the change when originally proposed. The committee decided there was no basis for the change as stated and unanimously voted to reject it.

A5 (Y = 3, YWR = 1, N = 1). After a brief discussion the committee unanimously rejected the proposed change for the same reasons stated under Item A4.

A6. This item was superceded by the letter ballot of May 5, 1980.

Category B items on the May 5 ballot were next considered. It was explained that these items required action by other committees and, depending on final action taken by Technical Committee 4, would be taken up by the Coordinating Committee at its meeting on June 5-6.

B1 (Y = 2, YWR = 2, A = 1). Mr. Fintel was asked to review the comments and response of Technical Committee 2 to proposed changes in the seismicity indices of Table 1-B. After discussing the potential impact of Table 1-B on concrete construction in various map areas, there was a consensus of the committee that some changes to Table 1-B were needed. It was agreed that changes in the original indices for map area 4 through 7 would not be appropriate. After a lengthy discussion of map areas 2 and 3, the committee unanimously approved the following motion:

"Recommend that Committee 2 alter the Seismicity Indices in Table 1-B, Chapter 1 to read as listed below and that Committee 2 carefully review map area 3 to determine whether or not certain areas such as New York City should more appropriately be assigned to a map area of 2 for concrete construction."

Map Area Number	Seismicity Index
7	4
6	4
5	4
4	3
3	2
2	1
1	1

B2. This item was superceded by the letter ballot of May 5, 1980.

B3 (Y = 1, YWR = 2, N = 1, A = 1). Mr. Fintel explained that Technical Committee 2 has already taken action that is responsive to the intent of this proposed change and, therefore, Item B3 is moot. Based on this, the committee unanimously agreed to withdraw the proposed change.

B4 (Y = 3, YWR = 1, A = 1). Mr. Fintel reported that Technical Committee 2 has recommended a change which responds to Item B4 and the issue is now moot. Thus, the committee agreed to withdraw the proposed change.

B5 (Y = 3, N = 1, A = 1). Mr. Fintel reviewed the action taken by Technical Committee 2 in recommending to the BSSC that a committee be established to study coupled shear walls and eccentrically placed frames. Technical Committee 4 unanimously agreed to withdraw the proposed change.

B6 (Y = 1, YWR = 3, A = 1). Mr. Salomone, Secretary of Technical Committee 3, reviewed the response of that committee to the proposed changes to Section 7.4.4 submitted by David Sheppard in his letter of December 21, 1979, to Technical Committee 4. Technical Committee 4 decided that the issues of dowels and exposed strand should be treated separately and took the following actions:

1. Unanimously recommended that the following be added to the second paragraph of Section 7.4.4.

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

2. Unanimously recommended that the following sentence be added to Section 7.4.4(E).

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

B7 (Y = 2, YWR = 1, N = 1, A = 1). The committee concluded that the proposed change was not clear as stated and unanimously recommended that the following wording be added to Section 8.2.2 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."

B8 (Y = 3, N = 1, A = 1). Discussion centered on what UBC requires for exterior wall attachments and whether it was appropriate to double the elastic forces rather than to modify the performance factor. The committee decided that the proposed change should be withdrawn and that Table 8-B should be modified by inserting the words "Connector Fasteners" indented and immediately under "Wall Attachments" with a corresponding C_c factor of 6.0. Mr. Fintel opposed the change.

B9 (Y = 4, A = 1). Mr. Fintel reported that Technical Committee 2 has taken this proposal under consideration and it was unanimously agreed to withdraw the original resolution.

At this point, Mr. Leyendecker outlined for the committee the schedule for completing the review and refinement of ATC 3-06.

It was moved by Mr. Holland and seconded by Mr. Jenny that Item 4 of the agenda be tabled in view of the need to consider the March 19, 1980 draft of Appendix A (ACI Standard 318-77) and the May 28 version of Chapter 11 prepared by Mr. Neville of the Portland Cement Association. In the discussion which followed, it was pointed out that certain items on the letter ballot of May 5, 1980, involved provisions of ATC 3-06 outside of Chapter 11 and would, therefore, have to be considered by the committee, regardless of what action was taken on Chapter 11. The original motion was then amended to table ballot items M1, M3, M4, M5, M9 and M12. The motion, as amended, passed unanimously and the committee then addressed the remaining issues on the May 5 letter ballot.

M2 (Y = 7, YWR = 1). Reservations regarding this proposed change were withdrawn and the proposed change was approved by unanimous vote.

M6 (Y = 7, N = 1). Discussion centered on the ability of flat plate construction (waffle slabs in particular) to share in resisting lateral loads when properly detailed. The committee unanimously agreed to delete the last sentence of the proposed change to the eighth paragraph of Section 3.6.3 which is then to read as follows:

> "The loading is cyclical, so static ultimate load capabilities 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 11.6.1."

M7 (Y = 7, N = 1). In discussing the proposed change, it was pointed out that there is no reason to check shear stresses if the procedure for design of slab-to-column connections is properly carried out. Mr. Wyllie stated that he would withdraw his negative vote, but that his reservation regarding the proposed change was to stand.

M8 (Y = 7, N = 1). The discussion centered on simply supported prestressed beams developing a hinge at midspan due to vertical accelerations and the fact that 0.5 Q_D would be an excessive requirement. Mr. Wyllie agreed to change his vote from "no" to "yes with comment," the comment being as stated on his May 5 ballot.

The committee adjourned for lunch.

1:35 PM EDT Meeting reconvened by Chairman Cohen.

It was moved by Mr. Holland and seconded by Mr. Fintel that the May 28 version of Chapter 11 and Commentary submitted by Mr. Fintel (see Attachment B) be adopted to replace the current Chapter 11 and Commentary of ATC 3-06. Mr. Fintel, who deferred to Mr. Neville, was asked to give a summary of the May 28 version, the reason for the proposed change, and how the May 28 version and referenced March 19 draft of Appendix A (ACI 318-1977) would affect previous ballot items and actions taken by the committee to date. Mr. Neville stated that the items contained in the letter ballots of March 27 and May 5 had been incorporated. Mr. Cohen noted that additional items contained in Mr. Bertero's memorandum to E. Cohen and E. Pfrang and distributed to members of the committee during the morning session would have to be considered by the committee if the May 28 version of Chapter 11 were adopted. Mr. Wyllie questioned the wisdom of adopting for trial designs a new set of provisions that are incomplete and have not been thoroughly reviewed. A lengthy discussion ensued concerning the evolution of Appendix A, the advantages and disadvantages in replacing the current Chapter 11 with the May 28 version, and reasons supporting the proposed change. The motion was put to a vote and carried 6 votes "yes" and 1 vote "no."

It was moved and seconded that the following resolution be adopted by Technical Committee 4.

> "Regardless of subsequent actions, it is the firm intent of this committee that the final version of Appendix A, with appropriate modifications, be incorporated in ATC 3-06 after completion of trial designs."

The committee adopted the resolution by unanimous vote. The Chairman then requested a motion for the following statement of appreciation.

> "This committee wishes to thank Professor Bertero for his dedicated work and many technical contributions over the past months."

The motion was seconded and unanimously approved.

The committee next considered the provisions of the May 28 version of Chapter 11 in light of action already taken on letter ballot items. The following changes to Chapter 11 and Commentary were moved, seconded and unanimously approved.

Section 11.4.1, paragraph (D) under "EXCEPTION:" Delete $3\sqrt{f'_c}\, b_o d$ and add $\left(1 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d$.

Add new Section 11.4.2 - FRAMING SYSTEMS. Wording to be identical to Section 11.5.2 except as follows:

First paragraph under "EXCEPTION:"

Second line - delete "ACI 318, Appendix A" and add "Section 11.4.1"

Fourth line - delete "and toughness" and add "stiffness, stability, durability, and energy dissipation capacity"

Last line - delete "Appendix A" and add "Section 11.4.1."

Second paragraph under "EXCEPTION:"

Last line - delete 1.0 and add 1.5.

Section 11.5.2

First paragraph under "EXCEPTION:"

Fourth line - delete "and toughness" and add "stiffness, stability, durability, and energy dissipation capacity"

Second paragraph under "EXCEPTION:"

Last line - delete 1.0 and add 1.5.

Section 11.5.3

Delete "WALLS AND" from section heading.

Delete first and second paragraphs.

Commentary - Section 11.4

Fourth paragraph - move to end of Section 11.5.4 of Commentary (Note: 11.5.3 should be 11.5.4).

Mr. Holland and Mr. Wagner left the meeting because of flight schedules and assigned their proxy to the Chairman.

The committee next considered the changes to the May 28 version of Chapter 11 recommended by Mr. Bertero in his undated memorandum to E. Cohen and E. Pfrang. This memorandum was distributed to the committee during the morning session and is included in these minutes as Attachment C. In the following, the page and item numbers are identical to those in the Bertero memorandum.

I. <u>CHANGES NEEDED IN CHAPTER 11</u>

<u>Page 1.</u> The committee adopted the following for reference documents.

Reference 11.1-ANSI/ACI 318-17 "Building Code Requirements for Reinforced Concrete," including proposed revision of Appendix A - "Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions," dated 19 March 1980, American Concrete Institute.

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

<u>Page 2.</u> Section 11.4.1. The committee adopted the following change.

First paragraph, first line:

Replace "Where Moment Resisting Frame Systems are..." with "Where Ordinary Moment Frames are..."

<u>Page 3.</u> Section 11.4.1(E). The committee adopted the following change.

Last line - delete "as torsion reinforcement." and add "to resist torsion at discontinuous edges."

<u>Page 3.</u> Section 11.5.2. The committee recommended that the following be added to the definition of BRACED FRAME, Chapter 2, page 37.

"In Chapter 11, reinforced concrete braced frames may be referred to as structural trusses."

<u>Page 3.</u> Section 11.5.3. This paragraph has been deleted.

<u>Page 4.</u> Section 11.5.3. This paragraph has been deleted.

<u>Page 4.</u> Section 11.5.4. The committee adopted the following wording for this section.

"All frame components assumed to be not part of the seismic resisting system shall have demonstrated capabilities satisfying Section 3.3.4(C) and shall conform to the requirements of ACI 318, Appendix A.8; except, the lateral deformation requirements of A.8.1 shall not apply. If nonlinear behavior..."

With regard to the following items in Mr. Bertero's memorandum (identified under I. CHANGES NEEDED IN CHAPTER 11), the committee determined that they should be sent forward to ACI Committee 318-Sub 10 for consideration.

Page 3. A.4.3.2
Section 11.5.1
Section 11.5.2

The committee agreed to the addition of a new Section 11.5.5 to Chapter 11 which reads as follows:

11.5.5 - RELATIVE FLEXURAL STRENGTH OF COLUMNS

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

Insert 11.7.2(A) of ATC 3-06 with the following changes.

First line after "joint" - insert "where framing columns resist a factored axial compressive force larger than $A_g f'_c$ /10 and in the plane..."

Third line from bottom - delete "Section 11.7.2(C)" and add "ACI Appendix A.4.4"

On page 3 of the Commentary, the committee agreed to add a paragraph and figure prepared by Mr. Hawkins which address reinforcement details at a discontinuous edge. The paragraph and figure are to be inserted after the third paragraph of Section 11.4.

The committee next considered those items listed under II. <u>CHANGES NEEDED IN THE NEW APPENDIX A TO MAKE IT CONSISTENT WITH THE ATC 3-06 PROVISIONS</u> in Mr. Bertero's memorandum. The following items were accepted by the committee as requiring a change in ACI Appendix A or as already accomplished through changes to the May 28 version of Chapter 11.

<u>Page 1</u>. A.0
<u>Page 2</u>. A.1
<u>Page 4</u>. A.2.1.4
<u>Page 8</u>. A.4.2.2
<u>Page 9</u>. A.4.3.2
<u>Page 9</u>. A.4.4.1
<u>Page 11</u>. A.5.3.1
<u>Page 14</u>. A.7.1.3

The committee agreed that all other Appendix A items identified in Mr. Bertero's memorandum should be sent forward to ACI Committee 318-Sub 10 for consideration.

The committee next took up unresolved items on the letter ballot of May 5, 1980.

<u>M10</u> (Y = 6, YWR = 1, N = 1). Discussion centered on the documentation supporting the proposed additions to Section 7.5.3. Specifically, the nature of the cyclic load tests was called into question. After extensive discussion it was moved and seconded that the proposed change as stated under Item M10 on the letter ballot of May 5 be withdrawn. The vote was Y = 1, N = 2, A = 2. Therefore, the change will be sent forward as originally stated.

M11 (Y = 7, N = 1). The committee discussed this issue at length, considering the possible reasons for the limitation in Section 7.6.1, documented damage to precast-prestressed piles subjected to seismic loads and the basis for removal of the limitation. It was moved and seconded to send the proposed change forward as stated under Item M11 on the letter ballot of May 5. The vote was Y = 3, N = 2. Therefore, the change will be sent forward as stated under Item M11.

M12. Because of actions taken by the committee up to this point in the meeting, this item was deemed to be moot.

To complete its action on the adoption of the May 28 version of Chapter 11 and references indicated therein, the committee conducted a letter ballot. The ballot item, designated as Y1, was stated as follows:

Y1. "Revise Chapter 11 and Commentary Chapter 11 of ATC 3-06 to read as per May 28, 1980 proposal, as modified in meeting of June 4, 1980, and changes necessary to incorporate those revisions into the remainder of ATC 3-06.

The results of the ballot were as follows:

Mr. Cohen "yes"	Mr. Prendergast "yes"
Mr. Fintel "yes"	Mr. Wyllie "no"
Mr. Hawkins "yes"	Mr. Jenny "yes"
Mr. Holland "yes"	Mr. Wagner "yes"

The committee next conducted a letter ballot regarding its intent to see ACI Appendix A incorporated in ATC 3-06. The ballot item designated as R1, was stated as follows:

R1. "Regardless of subsequent actions, it is the firm intent of this committee that the final version of Appendix A, with appropriate modifications, be incorporated in ATC 3-06 after completion of trial designs."

The results of the ballot were as follows:

Mr. Cohen "yes"	Mr. Prendergast "yes"
Mr. Fintel "yes"	Mr. Wyllie "yes"
Mr. Hawkins "yes"	Mr. Jenny "yes"
Mr. Holland "yes"	Mr. Wagner "yes"

There being no further business, the meeting was adjourned at 9:20 PM EDT.

Respectfully submitted,

R. D. Marshall Kyle Woodward

ATTACHMENT A

A G E N D A

Meeting of Technical Committee 4 - Concrete
National Bureau of Standards
Gaithersburg, Maryland
June 4, 1980

1. Purpose of meeting and ground rules.
2. Approval of minutes of previous meeting.
3. Resolution of negative votes and reservations on letter ballot of March 27, 1980.
4. Resolution of negative votes and reservations on letter ballot of May 5, 1980.
5. New items for consideration.
6. Approval of final committee recommendations.

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 Appendix A* - Requirements for Reinforced Concrete Building Structures Resisting Forces induced by Earthquake Motions, American Concrete Institute.

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.

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.

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

11.4.1 - ORDINARY MOMENT FRAMES

Where Moment Resisting Frame Systems 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 $3\sqrt{f'_c}b_o d$ 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 as torsion reinforcement.

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.

11.4.2 - FRAMING SYSTEMS

All components of the seismic resisting system (moment frames, structural walls, braced frames, and diaphragms) shall be proportioned in accordance with provision 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 and toughness 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.

11.5.3 - STRUCTURAL WALLS AND DIAPHRAGMS

Structural walls shall have vertical boundary members at wall edges as required by ACI 318, Appendix A.5.3.1. Vertical boundary members shall also be provided at any level of a structural wall where tensile axial forces can be developed.

Structural diaphragms shall have special transverse reinforcement as required by ACI 318, Appendix A.5.2.3. Special transverse reinforcement shall also be provided whenever tensile axial forces can be developed across the entire diaphragm section.

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 conform to the requirements of ACI 318, Appendix A.8; except that frame elements assumed not to be part of the lateral force resisting system shall have demonstrated capabilities satisfying Sec. 3.3.4(c). 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.

TABLE II-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 - <u>REQUIRED STRENGTH</u>

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 - <u>SEISMIC PERFORMANCE CATEGORY A</u>

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 - <u>SEISMIC PERFORMANCE CATEGORY B</u>

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.

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).

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 "toughness" requirement for framing systems not satisfying the requirements of ACI 318 Appendix A refers 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.

11.5.3 - 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.

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 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 inertia forces within the diaphragms to elements of the lateral-force resisting system.

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 3-8 - 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."

REASON: Chapter 11 is revised to reference the nationally recognized design standard, ANSI/ACI 318-77 "Building Code 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 II 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.

MEMO TO: E. Cohen, Chairman of Technical Committee 4: Concrete Review and Refinement of ATC 3-06, and
E. Pfrang, Chief of Structures and Material Division, NEL

FROM: V. Bertero, Representative of ATC

RE: Technical Implications of Incorporating ACI 318-77 and New Appendix A by Reference into ATC 3-06

According to the request formulated by you through Mr. Fintel's letter of May 29, 1980, I met with Mr. Fintel and Mr. Neville, ACI Committee 318 Secretary, on Friday, May 30, 1980, at 5 p.m. in 750 Davis Hall, University of California, Berkeley, to discuss the above technical implications. As requested in the same letter, the following are my written comments. It should be noted that these comments are of a preliminary nature as I did not have time to go through the document as thoroughly as I would like since it was only delivered to me on the evening of Wednesday, May 28, 1980. For example, the provisions regarding joints of frames (Section A.6 of the new Appendix A) differs considerably from the ATC provisions on joints (Section 11.7.3). To comment properly on the implications of this change would require the technical background material (data) on which the changes have been based and the time to study it. I did not have either.

I. CHANGES NEEDED IN CHAPTER 11.

Page 1. Sec. 11.1 should read: Refs. 11.1:

[1] ANSI/ACI 318-77 "Building Code Requirements For Reinforced Concrete" but excluding Appendix A; and

[2] New proposed Appendix A - Requirements for Reinforced Concrete Building Structures Resisting Forces induced by Earthquake Motions, 19 March, 1980.

Page 2. Sec. 11.4.1 should read "Where Ordinary Moment-Resisting Frame Systems are used for the seismic-resisting system, frame components (beams, columns, and their joints) shall be proportioned to satisfy, in addition to the requirements of Chapters 1 through 17 of Ref. [1] (ANSI/ACI 318-77), the provisions A.3.2, A.3.3, A.4.3 and A.8.2 of Ref. [2]. [NEW APPENDIX A] with the following additions and exceptions:

1. A.3.2.1 Last sentence should read "At least two No. 5 or larger bars shall be provided continuously both top and bottom."

2. A new section, A.3.2.5, should be added in the new Appendix A. This section A.3.2.5 should contain the provisions required in ATC Sec. 11.6.1, paragraphs 4 and 5, i.e., "A flexural member framing . . . yield stress." "Longitudinal reinforcement . . . for the reinforcement."

3. A.4.3.2 The first sentence should read "Lap splices are permitted only within the center half of the span and shall be proportioned as tension splices. Welded . . ."

Page 3. Sec. 11.4.1 (E). Add " . . . considering the probability of full reversals of the sense of the torsional moments (torsional resistance combined with flexural under reversal moments deteriorate significantly when conventional web reinforcement is used)."

Page 3. Sec. 11.5.1. Add at the end of this section ". . . A.2.5, except that ASTM 615 Grade 60 reinforcement should not be used when welding of this reinforcement is used." (See my comments of Feb. 11, 1980.)

Page 3. Sec. 11.5.2. First paragraph, last line should be changed as follows: ". . . provisions of Ref. [2], i.e., new proposed Appendix A." Second paragraph, second line, same change as above. (The same change should be made throughout the whole proposed draft.)
In the first paragraph it is necessary to clarify that ATC refers to "braced frames" while A.5 refers to trusses. This inconsistency should be removed. I recommend that, rather than incorporating the exceptions here, a new Section 11.6.1 be added on page 4 as it was recommended be done on the May 5, 1980, ballot, i.e., a new Section 11.9 of the ATC document.

Page 3. Sec. 11.5.3. Should read "Structural walls shall have vertical boundary members which shall be proportioned to satisfy the provision A.5.3 of the New Appendix. Vertical boundary . . . can be developed. If lap splices are needed at these levels, they shall be proportioned as tension splices."

Page 4. Sec. 11.5.3. First paragraph (top of page) should be changed to read "Structural diaphragms shall be provided with boundary or edge elements at any section where tensile axial forces can be developed across the entire diaphragm section. These boundary elements shall be designed as required by A.5.3. If lap splices are needed at these sections, they shall be proportioned as tension splices."

Page 4. Sec. 11.5.4. This section should read as follows: "STRUCTURAL COMPONENTS NOT PART OF THE SEISMIC-RESISTING SYSTEM.
All structural components assumed to be not part of the seismic-resisting system shall comply with Sec. 3.3.4(C) and shall conform with the provisions of Sec. A.8 of the new Appendix A except for Sec. A.8.1. This Sec. A.8.1 does not apply to the investigation of the deformation compatibility of these components; Sec. 3.3.4(C) is the one that should be used.
The design of such components shall satisfy the minimum reinforcement requirements specified in Chapters 7, 10 and 11 of ACI 318 and Secs. A.3.2.1 and A.5.2.1. 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 provisions A.3.3 and/or A.4.4 of the new Appendix A.

II. CHANGES NEEDED IN THE NEW APPENDIX A TO MAKE IT CONSISTENT WITH THE ATC 3-06 PROVISIONS

Page 1. A.0 Notation
h – should read h''
Note that some notations are different from those of ATC. For example, h'' is h_c in ATC, S is S_h in ATC, and P_i is P_n in ATC. Therefore it is

recommended the notations be reviewed thoroughly for Appendix A and ATC to assure their consistency.

Page 2. A.1 Definitions
There are discrepancies in some of the definitions used by ATC and Appendix A. For example, the definitions of cross-tie do not agree; also Structural Wall vs. Shear Wall, Structural Diaphragms vs. Diaphragm, Structural Trusses vs. Braced Frames, etc. Therefore it is recommended that the definitions in the two documents be thoroughly reviewed and the discrepancies removed.

Page 3. Definition of Anchorage Length for a Bar with a Standard Hook.
This definition does not agree with results of laboratory experiments and field inspection of damages. The effective length of anchorage cannot be counted from the critical section (where the strength of the bar which is located at the faces of the joint is to be developed). The concrete of the joint that is not confined (which has the shape of a cone) is not effective in supplying anchorage. This definition should be changed to consider the cone of unconfined concrete.

Page 3. Sec. A.2.1.1 This provision should be clarified. Limitations on the amount of energy dissipation that can be used, or would be acceptable or tolerable, should be specified. Can these provisions be used when the nonlinear response of the structure would demand "displacement ductility" of the order of 10, 20, 30, 40, 50, 60 . . . ? As is it written now, it is too vague and could lead to misuse of the provisions.

Page 4. Sec. A.2.1.3. Are the requirements for Zone 2 as defined by the UBC 1979 (I assume that it is the 1979 edition of the UBC to which this Appendix A refers) compatible with the requirements for good seismic performance of buildings assigned to Category B? This should be discussed and clarified.

Page 4. Sec. A.2.1.4. Are the requirements of the UBC 1979 for regions following Zones 3 and 4 sufficient to guarantee good seismic performance for buildings assigned to Categories C and D? This should be discussed and clarified.

Page 5. Sec. A.2.3.2. Does $\phi = 0.5$ apply only to the computation of the strength of the element under concentric axial force, or does it apply also to the combined axial force and bending moment, i.e., to the whole N-M interaction diagram for $N > A_g f'_c/10$ (as it was established in ATC)?

Page 5. Sec. A.2.5.1. A flag regarding the weldability of ASTM A615 Grade 60 should be inserted. Furthermore, it should be noted that, while ATC required that in tests the actual yield stress not exceed the specified yield stress by more than 21,000 psi (18,000 + 3,000), the new Appendix A allows 22,000 psi (18,000 + 4,000). I do not have the background material that has been used to justify this change. Note that the higher the value that is accepted, the less meaningful become the computations based on specified yielding (quality control of material is a must if we want to improve seismic-resistant design and construction).

Page 5. Sec. A.3.2.1. The last sentence should read "At least two No. 5 or larger bars shall . . ."

Page 8. Sec. A.4.2.1. This section should be modified to read as follows: "At any joint . . . the sum of the flexural strengths of the columns calculated considering the critical combination with the possible critical axial forces (whole range of possible axial forces acting in combination with the moments should be considered) shall exceed the sum of the moments at the columns obtained from the equilibrium at the joint when it is considered that the beams framing into that joint in the plane of the frame under consideration reached their flexural strength. The flexural strenghs shall be . . ."

Page 8. Sec. A.4.2.2. This section should be deleted or completely modified. Reasons: It allows the design of weak column-strong beam frames that can lead to soft story. Since the time this philosophy was proposed, I have opposed it because it leads to an unsound seismic-resistant system. It is not that the columns cannot be made ductile, but rather that the formation of a soft story leads to such large demands of energy dissipation capacity (ductility displacement demands) from the columns that these demands cannot be supplied. Therefore, it should be made clear that, except for frames of more than 2 stories, attempts should be made to prevent the development of soft stories. Any provision that will allow the formation of such soft stories should be deleted. Following this basic seismic-resistant guideline, if this section is not deleted it should be modified as follows: "A.4.2.2 - At any story level of a frame, a certain number of columns could be allowed to not satisfy Sec. A.4.2.1 provided that the remaining columns in that story of the frame complying with the requirements of Sec. A.4.2.1 are capable of elastically resisting the entire story shear at that level, accounting for the altered rigidities and torsion resulting from the omission of elastic action of the nonconforming columns. In addition, the nonconforming columns shall be provided with transverse reinforcement as specified in Sec. A.4.4 over their full height if the factored axial force in those columns exceeds $(A_g f'_c/10)$."

Page 9. Sec. A.4.3.2. At the end of the first sentence should be added ". . . span and shall be proportioned as tension splices. Welded . . ."

Page 9. Sec. A.4.4.1. In the list of notations, the following corrections should be made: Replace h with h", also in the definition of A_{s_h}. If this notation is used, the notation in ATC, pp. 40-43, should be modified also.

Page 10. Sec. A.4.4.1 Item (4). This item should be deleted as it can lead to unsound seismic-resistant practice by allowing columns without ductility since no confinement is required. Confinement of the concrete core is not only required for developing extra strength in the confined concrete required to compensate for the loss of the cover, but also to increase the deformation capacity (ductility). It is well documented through experiments and field inspection of earthquake damages that the cover of the columns at the joints will pull out and spall, reducing the effective area of concrete available to resist the internal forces to an effective cross-sectional area even smaller than that of the confined core. Application of the requirements of this Appendix does not guarantee that the column will remain elastic, because of the effects of strain hardening of beam reinforcement and the effects of higher modes of vibration. It is for these same reasons that I strongly support the recommendation in the present UBC (1979) that requires that shear strength of columns be computed based on the column core area.

The application of the provision of this section together with Sec. A.4.2.2 can lead to disaster. Therefore, I strongly recommend the deletion of these two sections or their modification.

Page10. Sec. A.4.4.4. At the end of this provision should be added "For members for which the calculated point of contraflexure is not within the middle half of their span, the special transverse reinforcement specified above should be provided over the full height of the members." (See ATC 11.7.2(B)5 (p. 106).

Page 11. Sec. A.5.2.3. What is understood by "elements of structural diaphragms" should be clarified. Are these Collector Elements and/or Boundary Elements? This should be specified. I also consider it necessary to add after the fifth line of this provision the following requirement: ". . . 0.15 f'_c, provided that no tensile forces or significant shear forces are developed simultaneously in these elements. If these elements could be subjected to significant shear forces (e.g., $v_u = 3\sqrt{f'_c}$) and to tensile forces, they shall have special transverse reinforcement as specified in Sec. A.4.4 over the total length of the element.

Page 11. Sec. A.5.3.1. The requirement should be added for the case where tensile axial forces can be developed (see 11.5.3).

Page 13. Sec. A.6.3.1. This whole provision needs clarification.
(1) It is suggested that the definition of A_j be given in the notation, Sec. A.0, or directly in this section rather than giving it in Sec. A.6.3.2. Furthermore, the definition given is not clear. What does "the design shear commentary force" mean? Should this read "shear generating force"? Should A_j be the total area, the effective area bd, or the confined core area?
(2) In lines 2 and 5 the symbol γ is missing; they should read "coefficient γ".

Personally, I question the soundness of some of these provisions (see my general comments about weaknesses in the ATC and Appendix A provisions).

Page 13. Sec. A.6.4. This section needs clarification. The value of ϕ is not given in this section. The reader has to go to the Commentary to find that i has been defined in Sec. A.2.3.3. No indication is given of the location of the critical section for computing the development lengths ℓ_{ah} and ℓ_{as}. I personally would like to see explicitly in the equation for the estimation of the anchorage length the 1.25 f_y. This is a new section which appears able to give quite different results than those obtained according to the recommendations of Committee 352 (ACI Journal/July 1976), depending on where the critical section for anchorage is taken. I did not have the background material at hand to study this new section, but it appears to me these provisions do not properly consider the effects of deformation reversals to which the anchored bar can be subjected. The reasons follow.

(1) The commentary refers to data presented by ACI Committee 408 which does not include the effect of deformation reversals. Apparently the only attempt to account for this effect has been to specify a reduction factor of $\phi = 0.65$ rather than the $\phi = 0.80$ recommended by Committee 408. This is again a misuse of the original intent of the reduction factor ϕ.

(2) No indication is given where the critical section for anchorage should be located. The research I have conducted clearly shows that there is a core of unconfined concrete [whose depth depends on cover (shell concrete) and spacing of reinforcement in the joint core] which is ineffective in developing the reinforcement. Thus it appears to me that, if designers assume that the critical section is at the face of the joint, the application of this provision A.6.4 can lead to unconservative anchorage, particularly in the case of narrow columns.

Therefore at present I cannot support or recomment the inclusion of this provision.

Page 14. Sec. A.7.1.2. Although this section is similar to that in ATC 11.7.2(C), p. 106, I believe it is incorrect. The nominal moment strengths should be calculated for the critical axial force in the possible range of axial forces. In the selection of this critical axial force, proper N vs. M interaction diagram and the variation of the shear strength with N should be considered.

Page 14. Sec. A.7.1.3. This section cannot be used in conjunction with the ATC document. The design shear force shall be obtained from the factored loads and combinations of Sec. 3.7 of the ATC document, and not from Sec. 9.2 of ACI 318.

Page 15. Sec. A.7.3.1. The application of equation (A-5) to barbell and flanged wall cross sections is not clear because, according to the definitions of A_c and ρ_a, only the areas of concrete and steel bounded by web thickness and height of section should be considered. It appears to me that all the steel located in the edge member of the barbell shape should be considered. Similarly, all the steel located in the flange effective width of the flanged cross section should be considered.

Page 17. Sec. A.9.2.2. Equation (A-6) does not agree with equation 11-6 of ATC. Note that in (A-6) the reduction factor ϕ is missing. This appears contrary to the main philosophy of the whole ACI 318-77 document in which Required Strength $\leq \phi$ [Nominal Strength]. Furthermore, notation for the factored compressive force at the construction joint, i.e., P_j, in ATC is P_n. Therefore, a change should be made either in Sec. 2.2 Symbols of ATC or in A.0 and A.9.2 of the new Appendix A. Note the inconsistency in A.9.2 regarding the notation of this force. In equation (A-6) this force is designated as P_j but three lines below this equation (A-6) it is defined as P_n.

Summary of Committee 4 Action

on

July 16-17, 1980

Those members who were present are as follows:

Name	Representing
Neil Hawkins (Acting Chairman)	Post-Tensioning Institute
Daniel Jenny	Prestressed Concrete Institute
Loring A. Wyllie, Jr.	Structural Engineers Association of California
James Prendergast	Interagency Committee on Seismic Safey in Construction
S. K. Ghosh (alternate)	Portland Cement Association
James Lefter	Building Seismic Safety Council
Richard Marshall (Secretary)	National Bureau of Standards
Kyle Woodward (Secretary)	National Bureau of Standards

At the request of Mr. Cohen, Committee Chairman who could not be present, Mr. Hawkins served as Acting Chairman of Committee 4 and served as the Committee Spokesman during the Joint Committee meeting.

The changes to ATC3-06 recommended by Committee 4 were presented to the Joint Committee by the Acting Chairman July 16. There was considerable reaction by the Joint Committee to several of the proposed changes especially those pertaining to the issue of a revised ATC3-06 Chapter 11 referencing the draft version of ACI 318 Appendix A.

A meeting of those committee members present (5 out of 8 voting members) was called in the afternoon of July 16 to discuss the implications of the Joint Committee's reaction to the revised Chapter 11. The committee agreed that the single ballot item including so many proposed changes was a handicap to the adoption of particular revisions unanimously endorsed by the committee. It was agreed by the committee that if the ballot item on the all inclusive revised Chapter 11 was defeated by the Joint Committee, then Committee 4 would request that the Building Seismic Safety Council (BSSC) permit a restructuring of the ballot item and resubmission on the BSSC ballot in the Fall of this year. The restructuring would involve the separation of each individual proposed revision included in the overall ballot item as an individual ballot item.

The actions of the Joint Committee on the following day (July 17), however, permitted Committee 4 to submit additional items to the Joint Committee for inclusion on the Joint Committee's letter ballot. The instructions to the committee were such that the additional items had to have been directly discussed and balloted by Committee 4 in its previous meetings. The acting chairman, upon discussion with the chairman, directed the secretaries to prepare the additional ballot items. The ballot items addressed the particular issues included in the overall ballot item covering the adoption of the revised Chapter 11. It was felt that the votes on each of the separable issues (e.g. flat slabs) would be helpful to the BSSC members in ascertaining the level of support for the proposed revisions. Such information would not be present from the vote total on the single overall ballot item.

The additional ballot items were prepared and submitted to the Acting Chairman and Mr. Sharpe of ATC for comments. After review by each, the items were sent to the Joint Committee for balloting. See the attachments for the additional letter ballot items submitted to the Joint Committee.

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."

3.2 Committee Roster

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(representative on Committee 2: Structural Design) - Fintel

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3.3 Selected Committee Correspondence and Applied Technology Council Comments

1. Letter from Mr. Sheppard to Committee 4, December 21, 1979 - 4 pages

2. Letter from Mr. Fintel to Sec. Committee 4, January 8, 1980 - 13 pages

3. Letter from Mr. Forell to Sec. Committee 2, January 11, 1980 - 2 pages

4. Letter from Mr. Bertero to Sec. Committee 4, January 31, 1980 - 9 pages

5. Letter from Mr. Manning to Committee 4, January 31, 1980 - 5 pages

6. Mr. Bertero's comments on revisions proposed by Mr. Manning, February 11, 1980 - 2 pages

7. Letter from Mr. Cohen to Sec. Committee 4, February 11, 1980 - 1 page

8. Letter from Mr. Hawkins to Sec. Committee 4, February 26, 1980 - 16 pages

9. Letter from Mr. Sheppard to Committee 4, March 25, 1980 - 2 pages

10. Mr. Bertero's comments on revisions proposed by Mr. Hawkins, April 7, 1980 - 4 pages

11. Letter from Mr. Sheppard to Committee 4, April 2, 1980 - 4 pages

12. Memorandum from Mr. Bertero to Committee 4, June 2, 1980 - 2 pages

13. Memorandum from Mr. Bertero to Mr. Cohen and Mr. Pfrang (undated-distributed to Committee 4 on June 4, 1980) - 7 pages

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December 21, 1979

TO: Dan Jenny
Edward Cohen
Eugene D. Holland
Joe Manning
Jim Prendergast
Mark Fintel
Gene Corley
Neil Hawkins
Vitelmo Bertero
Jim Lefter
Richard Marshall

RE: Proposed Revisions to ATC 3-06

Gentlemen:

In accordance with instructions given by Committee Chairman Ed Cohen, I have submitted for your consideration proposed revisions to the document written in code language, with appropriate reasons for each. Please note that these proposed revisions must be reviewed by the Technical Activities Committee of the Prestressed Concrete Institute before they become our official industry position; however, I am not aware of any conflicts in philosophy at this time.

<u>SEC. 11.2</u> Revise the second sentence in Section 11.2 to read as follows: "Precast and/or prestressed concrete components may be used if the resulting construction complies with the requirements of Sec. 3.6 and this chapter, except as specifically modified in Sections 11.9, 11.10, 11.11 and 11.12."

BASIS: Present provisions of ATC 3-06 exclude the use of prestressed concrete (by omission); specific subsections should be established for the unique and separate design characteristics of precast and prestressed concrete.

SEC. 11.2 Revise capacity reduction factors for connections of precast components to read as follows:

Connection capacity as governed by concrete: $\phi=0.65$
Connection capacity as governed by steel: $\phi=0.90$

BASIS: Conservative industry guidelines recommend a 3/4 factor to be used in connection design for concrete shear. This results in a value of $0.75 \times 0.85 = 0.65$. Chapter 10 indicates a value of $\phi = 0.9$ for steel

SEC. 11.8 Revise the last sentence of paragraph 11.8.2 to read as follows: "Diaphragms for precast concrete floor systems may be developed with cast-in-place concrete topping, shear friction boundary reinforcing, or properly designed component connectors, either welded or grouted."

BASIS: Current provisions exclude the use of untopped precast or prestressed concrete floor systems.

SEC. 11.8 Revise the first sentence of Section 11.8.4 to read as follows: "Boundary members shall be provided as required by Section 11.8.1 and 11.8.2, except for large panel precast concrete systems building construction with energy dissipating mechanisms formed in coupling links, as indicated in Section 11.11."

BASIS: Research and testing conducted at MIT and by Yugoslavs, Japanese and Russians.

SEC. 11.9 to 11.12 Add the following new sections to Chapter 11:

11.9 Plant Cast Prestressed Concrete
11.10 Post-Tensioned Concrete
11.11 Plant Cast Precast Concrete
11.12 Site Cast Precast Concrete

BASIS: Provisions for precast and prestressed concrete are currently scattered throughout the document. Requirements in design sections should be performance oriented, applicable to all materials; specifics for prestressed or precast concrete should be covered in the above sections. See also my letter to William W. Moore presented at the 1st Annual Building Seismic Safety Council Meeting on November 8, 1979.

SEC. 11.2 Add at the end of the sentence reading "Axial compression or axial. . . . for the full height of the component.": "Non lateral load resisting compression members designed in accordance with Sections 11.11 or 11.12 shall have capacity reduction factors as given in ACI 318-77.

BASIS: The arbitrary assignment of a low capacity reduction factor for "pin-ended" compression members is not warranted, when top and bottom connections are designed to accommodate maximum drift movements and increased bending moments induced by the P-Δ effect.

SEC. 3.3.4 In Section (A) 3. - Delete the last sentence reading "This system is limited to buildings not over 240 feet in height."

BASIS: Assignment of height limitations on these structures is arbitrary and inconsistent with actual performance of these structures. Proper building design and location of stiffening elements should govern as is alluded to in Section 3.4.1. The design requirements should be performance oriented, and not consist of arbitrary requirements.

SEC. 3.3.4 Add new type (A) 4: "Coupled shear wall systems with primary inelastic action along these vertical coupling elements providing energy dissipation."

BASIS: The work of Becker at MIT.

SEC. 3.3.5 Delete the second sentence in this section in its entirety.

BASIS: Arbitrary height limitation; see above.

SEC. 7.4.4 Revise Section 7.4.4(E) to read as follows: "The upper 2 feet . . . . or equivalent spirals. The pile cap connection may be made by developing exposed strand or by the use of field placed anchor dowels grouted into sleeves cast in the pile top as outlined in Section 11.9.

BASIS: Present accepted practice in UBC-79 and CALTRANS specifications.

SEC. 7.5.3 Revise the last sentence of Section 7.5.3(c) to read as follows: "Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curveatures resulting from a dynamic analysis of the soil profile present, with detailing as specified in Section 11.9."

BASIS: See my letter and accompanying documentation from Gerwick, et al presented at the BSSC meeting on November 8, 1979.

SEC. 7.6.1 Revise this section to read as follows: "All piling types in Category D shall be designed to withstand maximum imposed curveatures resulting from a dynamic response analysis of the soil profile present."

BASIS: Same as 7.5.3 above. Foundation requirements should be performance oriented, and not arbitrarily penalize certain materials (prestressed concrete) because of local bias, in spite of recent tests and successful design applications developing large curveatures resulting from layered soil movements in maximum credible seismic conditions.

SEC. 8.2.3 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.

SEC. 8.2.2 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.

Detailed provisions for Sections 11.9, 11.11, and 11.12 will be develpped later. I am assuming PTI will develop material for Section 11.10.

Very truly yours,

David A. Sheppard

David A. Sheppard
California Marketing Director

DAS:rd

PORTLAND CEMENT ASSOCIATION

5420 Old Orchard Road, Skokie, Illinois 60077 Area Code (312) 966-6200

January 8, 1980

Mr. Richard Marshall, Secretary
Technical Committe 4, Concrete
Tentative Seismic Provision Project
B168, Bldg. 226
National Bureau of Standards
Washington, D.C. 20234

Dear Mr. Marshall:

Enclosed are four proposed changes of provisions in ATC3-06 relating to concrete buildings.

Would you kindly transmit the proposed changes that also relate to other committees to the relevant groups.

Yours very truly,

Mark Fintel
Director Advanced Engineering

MF:gh

cc: Mr. Edward Cohen - American Concrete Institue
Mr. Eugene D. Holland - American Society of Civil Engineers
Mr. Joseph Manning - Concrete Reinforcing Steel Institute
Mr. James D. Prendergast - Interagency Committee on Seismic Safety in Construction
Professor Neil Hawkins - Post Tensioning Institute
Mr. David A. Sheppard - Prestressed Concrete Institute
Mr. Victor Bertero - Applied Technology Council
Mr. James Lefter - Building Seismic Safety Council

Submitted by
Portland Cement Association
Mark Fintel
January 1980

TABLE 1-8 – Page 35

REVISE THE "SEISMICITY INDEX" COLUMN OF TABLE 1-8 TO READ AS SHOWN BELOW:

TABLE 1-8

Coefficients A_a and A_v and Seismicity Index

Coeff. A_a Figure 1	Map Area Number	Coeff. A_v Figure 2	Seismicity Index
0.40	7	0.40	4
0.30	6	0.30	4
0.20	5	0.20	~~4~~ 3
0.15	4	0.15	~~3~~ 2
0.10	3	0.10	~~2~~ 1
0.05	2	0.05	~~2~~ 1
0.05	1	0.05	1

REASON: The seismicity indices were introduces 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.

It is not acceptable to require arbitrarily the same level of ductility detailing for acceleration levels of .40 (map area 7) as for acceleration level 0.15 (map area 4).

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.

Regarding the acceleration levels of 0.15 and 0.20, (map areas 4 and 5), the major question is which structural members will be yielding and how much ductility will be required in them. 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. Added ductility requirements should be imposed only if seismicity vs ductility correlation studies for map areas with acceleration levels of 0.10, 0.85 and 0.20 indicate levels of ductility demands requiring such detailing.

Submitted by
Portland Cement Association
Mark Fintel
January 1980

SECTION 3.3.4 - Page 46

DELETE SECTION 3.3.4(A)3

REASON: The height limitations are arbitrary, unjustified by today's level of knowledge of structural response and element strength and ductility, and should be removed. The best performer in reinforced concrete, the shear wall-frame interactive system, is limited to a height of 240 ft. In comparison, the special moment frame, which in reality becomes unbuildable at about 15 - 18 stories, is the only concrete system allowed above 240 ft. We do not believe that the reasons and circumstances which prevailed in the early sixties and led to similar height limitations (primarily a lack of knowledge) are still valid today.

To assure safety of multistory buildings above a height of 240 ft they may be analyzed and designed by the more realistic inelastic procedures, to make certain that ductility demands are within available limits. The only limiting factors to determine the height of buildings should be member capacity for strength and ductility and the overall response of the structure. Also, the rigidity of the structure should be considered in limiting the interstory distortions, thus assuring realistic damage control of nonstructural elements. In structures so designed <u>all</u> the earthquake forces and deformations are resisted by the elements of the structure in accordance with their relative strength and rigidities. To assure stability of inelastic structures, the inelastic procedure permits control of inelasticity so it may be confined to horizontal elements only, while assuring elastic behavior of columns or walls at all times if the designer so chooses.

As Corrected on 1/24/80

Submitted by
Portland Cement Association
Mark Fintel
January 1980

Section 3.9 - Page 51

ADD A NEW SECTION 3.9 TO READ AS FOLLOWS:

Sec. 3.9 - ALTERNATE INELASTIC DESIGN PROCEDURE

3.9.1 - INELASTIC RESPONSE HISTORY ANALYSIS

Structural systems may be analyzed by multi-degree-of-freedon inelastic response history analyses using appropriate earthquake records (adjusted in intensity to local seismicity). Resulting base shear shall not be less than 90% of that required by Eq. (4-1) of Chapter 4.

Such systems shall have members (beams, columns and/or structural walls) designed for resulting forces and deformations, as required by the inelastic analysis.

Interstory distortions (story drift) as computed by the inelastic response history analysis shall not exceed the allowable story drift Δ_a obtained from Table 3-C for any story.

3.9.2 - STRENGTH AND DUCTILITY

In proportioning of members, elastic strength or inelastic strength and ductility, as required by the analysis, shall be used.

Members of the structural system shown by the analysis to remain elastic during the response shall be proportioned as required by Sec. 11.6.

Members of the structural system shown by the analysis to undergo inelastic deformations during the response shall be proportioned as required by Sec. 11.7 for structural frames, and Sec. 11.8 for structural walls.

Maximum rotational ductility required by analysis of members proportioned according to Sec. 11.7 and 11.8 shall not exceed:

6 in beams with a span-to-depth ratio larger than 4

12 for beams with diagonal reinforcement having span-to-depth ratio of less than 2½

3 in shear walls with a height-to-depth ratio of more than 2 and having vertical boundary elements

3 for columns

REASON: Elastic static analysis cannot adequately determine forces and deformations in inelastic structures. Designing on the basis of elastic analysis may lead to insufficient ductility in some members, and to inadvertent shear failures, such as observed in the Banco de America building in the 1973 Managua earthquake.

The concept of ductility within the design process, which is in accordance with current practice, was developed on the basis of studies of single-degree-of-freedom systems. System displacement ductilities of 4 to 6 were utilized for a 1940 El-Centro type earthquake. However, in designing a structure, we deal with the ductilities of individual members, and not with overall system ductility. The relationship between the two may be different for each member in a structure, and changes in structural configuration will result in changes in the individual member ductilities. Therefore, while we are talking about system ductilities of 4 to 6, we may be faced with member rotational ductilities considerably larger, depending on the structural configuration, and strength and stiffness relationships. No systematic studies have been carried out to determine the distribution and magnitude of member ductilities within a structure. Consequently, in the present implementation of the overall concept to assure safety against brittle failures, we must, of necessity, provide maximum ductility in all columns, beams, and their connections, whether needed or not. In reality, from experience in earthquakes, and from inelastic analyses, it is known that ductility is not required in all members of a frame. Unfortunately, this important economic consideration is not included in ATC-3.

The major drawback of elastic analysis when applied to inelastic structures is that it does not allow us to determine the amount and distribution of ductility throughout the structure. We hope that the details specified in ATC-3 and other seismic codes will assure availability of the required ductility in all members which may become inelastic. We hope -- we do not know for sure.

Other shortcomings resulting from the use of elastic analysis for an inelastic structure are the possibility of inadvertent yielding of columns during very severe earthquakes, with its consequent effects on overall structural stability, and also the lack of an active control over the sequence of yielding during seismic response.

A procedure based on inelastic analysis needs to be introduced as an <u>alternate approach</u> for multistory buildings. Such a procedure became practicable with the development in recent years of highly efficient two-dimensional response history analysis computer programs. A good example of such a program is DRAIN-2D, developed at the University of California, Berkeley. The dynamic response is determined in the program by using a step-by-step integration of equations of motion. Inelastic characteristics of structural elements of both concrete and steel have been incorporated by the University of California, Berkeley (concrete) and by the University of Michigan (steel), respectively.

The explicit inelastic dynamic analysis design procedure entails the following steps:

1. Preliminary layout and design of the structural system based on gravity load requirements, code wind loading and code earthquake loading.
2. Modelling the structure for dynamic analysis in each of the two orthogonal directions. Frames and shear walls are to be lumped into the least number of vertical lines; the masses are concentrated at floor levels.
3. Selection, on the basis of local seismicity and of structural and soil characteristics, of design accelerograms with a potential to critically excite the structure.
4. Determination of forces and deformations in the members under the design earthquake, using inelastic response history analysis. A number of runs are required to choose the optimal combination between strength and ductility.
5. Proportioning of members for strength and deformability in the elastic and post-elastic ranges, based on resistances and ductility capacities known from tests.
6. Checking that the structure has enough ductility to survive, without collapse, the maximum credible earthquake possible at the site.

This alternate inelastic approach gives the design engineer a valuable tool for designing multistory structures in which the amount and distribution of inelasticity during the response can be controlled by the choice of strength relationships; consequently, ductility details can be included where they can be best utilized. Other advantages include the ability to:

- Design into the yielding members of the structure a desirable balance between strength and ductility.
- Predetermine a sequence of plastification so that energy can be dissipated without endangering stability. An early onset of yielding in beams limits buildup of axial loads in columns, of column moments and of shears in beam-column joints.
- Select perfromance criteria for the design earthquake (i.e., to have yielding beams and elastic columns), thus providing better damage control.
- Devise innovative and more effective structural configurations to dissipate seismic energy--systems we have not yet been able to devise because we have lacked the means to analyze them.

A number of design examples, carried out for shear wall-frame interactive systems, and for coupled wall systems, show the feasibility and technical superiority of the solutions, as well as the economic advantages of the inelastic approach. The procedure is applicable to both reinforced concrete and structural steel highrise structures.

The procedure is limited to fairly symmetrical structures for which a two dimensional model can be used with a degree of confidence.

Conclusion

While the present code provisions are of necessity overly conservative with respect to distribution of ductility, new procedures have recently become available which result in more rational and more economical structures.

Explicit inelastic response history analysis permits an alternate approach (based on energy dissipation considerations) that is applicable to multistory building structures of reasonably regular layout, for which inelastic dynamic analysis appears to be warranted. This inelastic response history analysis makes it possible to analyze a structure and to proportion its members for an optimum balance between strength and ductility, and to provide ductility details in all parts of the structure which are designed to undergo inelastic deformations. Obviously, information on available ductility of the members in question is a prerequisite.

The objective of the procedure is to establish a sequence of energy dissipating mechanisms and thereby impose on the structure a desired response, permitting no alternate types of behavior. A structure so designed is then detailed for ductility only in the predetermined hinging regions; this results in a more economical and technically superior structure.

Submitted by
Portland Cement Association
Mark Fintel
January 1980

TABLE 3-8 - Page 52

PROPOSED PROCEDURE TO DEVELOP RATIONAL VALUES FOR RESPONSE MODIFICATION COEFFICIENTS, R.

Response modification factors, R, introduced in ATC-3-06, are a significant departure from the previous K-values, and may have a serious impact on the construction industry. The concept of response modification factors, R, ranging from 1¼ to 8 to account for energy dissipation due to inelasticity and damping of the various structural systems and materials is conceptually clear, simple, and easy to apply, and represents a significant improvement over the present use of K-factors. However, the apparently arbitrary selection of R-factors in Table 3B, without studying their effect on member ductilities, makes the practical application of the concept very questionable. Since the overall underlying concept is a balance between strength and ductility, if the R-values lack a correlation with member ductilities, they are not much superior to the previously used K-values. A major uncertainty of the arbitrarily chosen R-values is the question whether the member ductilities actually available meet the ductility demands generated during an earthquake. Viable "R" values which answer this question can only be derived by means of inelastic response studies.

To evaluate the suggested arbitrary Response Modification Factors, R, of various individual systems and materials by comparing them with the previous "K" values (also unsubstantiated and adopted arbitrarily 40 years ago) is like the blind leading the blind.

Studies to determine realistic R and C_d values must be carried out for the various structural systems and materials listed in Table 38. The value of R to be derived from response history analyses is the ratio of base shear for the undamped elastic system to the base shear for the damped inelastic system, both systems representing the same structure, and both being subjected to a properly selected ground motion. The inelastic response history analysis of the damped inelastic system (designed by the R-factor approach) would yield required member ductilities corresponding to the assumed R-factor. If these required ductilities are attainable with the specified detailing, then the R-factor is realistic; otherwise it needs revision.

The following is a suggested procedure to derive R-values for a given structural system:

1. Prepare a preliminary design based on gravity loads and the traditionally Code-specified earthquake forces.
2. Prepare a 2-dimensional mathematical model of the structure, with masses concentrated at floor levels; use lumping to minimize the number of vertical lines.
3. Determine the fundamental period of the elastic structure, and assume a certain period change due to inelasticity during response.
4. Select from the library of accelerograms one, or several, records having a broad-band velocity response spectra potentially damaging to the given structure, considering the initial and lengthened periods. Normalize the records to a given intensity.
5. Run a response history analysis for the undamped elastic response. Determine the base shear, V_{el}.

6. Divide the base shear, V_{el}, by the assumed "R" (as given in Table 38 for the given system) and distribute the resulting base shear, V, over the height of the structure. Using a static elastic analysis, determine forces and proportion the members.

7. Run an inelastic response history analysis for the model in (2), using strength of members as determined in (6), customary damping values, and proper hysteretic models (for steel or concrete). Use the same imput motion record as in (5). Determine base shear, V_R.

8. If the base shear V_R is not the same as V in (6), adjust the strength of the members in proportion of V/V_r, and repeat step 7.

9. Determine the rotational ductility demands of all members. If these required ductilities are attainable with the specified detailing, then the prescribed R-factors are realistic; otherwise, they need revision.

The total effort required to determine practical numbers for R is extensive. However, it must be undertaken and systematically carried out if the proposed ATC-3-06 design provisions are to be based on a solid foundation.

FORELL / ELSESSER ENGINEERS, INC.

Forell · Elsesser · Chan Structural Engineers

Nicholas F. [illegible] SE
Eric Elsesser SE
F.C. Chan SE

Donald [illegible] Chappell SE
William [illegible] Honeck SE

January 11, 1980

James Harris, Secretary
Technical Committee No. 2
Tentative Seismic Provisions Project B168
Building 226
National Bureau of Standards
Washington, D.C. 20234

Gentlemen:

The enclosed lists of comments and recommendations are intended to improve on the Tentative Provisions to be used in the trial design test program.

The list prepared by me had assistance from members of the Structural Engineers Association of Northern California and was briefly discussed in a meeting of the Steering Committee of the Seismology Committee of SEAONC. The list prepared by T. Zsutty, Chairman of the State Seismology Committee, and Ed Zacker, past President of SEAONC, are transmitted as received.

I wish to restate my expressed concern at the December 11th meeting at the National Bureau of Standards. The importance of the Tentative Provisions is too great to limit the time for the preparation of comments and recommendations as severely as the schedule demands. The result of placing such a severe time restraint on this process will be a lingering doubt in the minds of the participants and their sponsoring organizations that they have not been given a fair opportunity to have their voices heard. I sincerely hope the door will not be closed for future well reasoned and sincere comments.

Very truly yours,

Nicholas Forell

/cs

encl. — incomplete for this copy

cc: Steve Johnston

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Section 11.8.1., 2. and 4. Need clarification that edge members or chords are required whenever tension stress exists in walls and diaphragms. These members must have ductile ties when computed gross section stress exceeds $0.2f'_c$, and only this requirement for ductile ties is discontinued when stress falls below $0.15\ f'_c$; but the edge member may be required for tension resistance beyond this level.

Section 11.8.4. Boundary Members: Clarify last paragraph.

Chapter 5. Dynamic Analysis needs to be completely redone.

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(1) It is an omission in the "Provision" that no reference is made to the use of seismic separation joints as a device to eliminate irregularities in building shapes. (Comment by N. F. Forell)

UNIVERSITY OF CALIFORNIA, BERKELEY

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COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
DIVISION OF STRUCTURAL ENGINEERING
AND STRUCTURAL MECHANICS

BERKELEY, CALIFORNIA 94720

January 31, 1980

Mr. Richard Marshall, Secretary
Technical Committee 4, Concrete
Tentative Seismic Provision Project
B168, Bldg. 226
National Bureau of Standards
Washington, D.C. 20234

RE: Review and Comments on the Proposed Revision of Chapter 11 of the ATC 3-06

Dear Mr. Marshall:

Enclosed you will find my review and comments on the revisions proposed by Dave Sheppard on behalf of the Prestressed Concrete Institute; Nicholas Forell, the SEAONC representative; and Mark Fintel, the PCA representative. In some cases it has been difficult to make comments because there were no specific proposed revisions and/or there was a lack of supporting evidence and reasoning. However, I think I have reviewed and commented on all the revisions that I have received so we have some basis for discussing the suggested changes more thoroughly at our February 21 meeting.

Sincerely yours,

V. V. Bertero
Professor of Civil Engineering

VVB/elm
Encl.

NOTE: Please transmit copies of this review to the members of committee 4 and to other appropriate technical committees

PROPOSED REVISIONS TO ATC 3-06

REVISIONS PROPOSED BY:

I. David A. Sheppard, Representative of the Prestressed Concrete Institute

1.1 SEC. 11.2 Revise the second sentence in Section 11.2 to read as follows: "Precast and/or prestressed concrete components may be used if the resulting construction complies with the requirements of Sec. 3.6 and this chapter. except as specifically modified in Sections 11.9, 11.10, 11.11 and 11.12."

This will not be needed as it will be part of the chapter.

COMMENT: This revision as proposed cannot be introduced until the proposed new sections, 11.1 through 11.12, are developed and further studies regarding adequate values of R and C_d for these precast and prestressed concrete systems are conducted. A modified revision is proposed.

REASONS: The requirements established in Chapter 11 cannot be considered independently, as they are the consequence of the structural design requirements (Chapter 3) which include the accepted analysis procedures (Chapters 4 and 5). In developing the requirements of Chapter 11, attempts were made to assure that the basic design equation, i.e.,

$$\text{DEMAND of} \left\{\begin{array}{c}\text{Strength}\\ \text{Stiffness}\\ \text{Stability}\\ \text{Energy Absorption}\\ \text{and}\\ \text{Energy Dissipation}\\ \text{Capacities (Ductility)}\end{array}\right\} \leq \text{SUPPLY of} \left\{\begin{array}{c}\text{Strength}\\ \text{Stiffness}\\ \text{Stability}\\ \text{Energy Absorption}\\ \text{and}\\ \text{Energy Dissipation}\\ \text{Capacities (Ductility)}\end{array}\right\}$$

is satisfied. Because of the substantial uncertainties involved in the current methods of estimating the DEMANDS, it is a good policy in seismic-resistant design to be generous in the SUPPLY, a philosophy which has been adopted in developing the requirements of Chapter 11. The estimation of the DEMAND is based, among other factors, on the use of the response modification factor, R. In the selection of the R values for R/C systems, besides the examination of the research data available regarding the seismic behavior of the systems, special consideration was given [as explained in the commentary of Section 3.3 (pp. 336-338)] to the observed general performance of these systems during past earthquakes: the general toughness (ability to absorb energy without serious degradation under reversals of deformations i.e., stable hysteretic behavior that guarantees good dissipation of energy); and the general amount of damping present in the system when undergoing inelastic deformation.

With this in mind, it should be noted that the R values given in Table 3B for R/C systems (7 for special moment frames, 5 1/2 for shear walls, 8 for dual systems, etc.) have been selected on the basis of the observed seismic performance and field and laboratory experimental data available on structural concrete systems designed and constructed on the basis of the present techniques for MONOLITHIC CAST-IN-PLACE ORDINARILY REINFORCED CONCRETE CONSTRUCTION. Because at present there is a lack of reliable information and experience regarding the seismic behavior of buildings with structural concrete systems based on the use of precast and/or prestressed components [see Proceedings of a Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, July 1977, Berkeley, CA, and Bertero, V. V., "Seismic Behavior of Structural Concrete Linear Elements (Beams, Columns) and their Connections," CEB Bulletin No. 131, AICAP-CEB Symposium, Rome, May 1979, pp. 123-313], it is recommended that the R and C_d values given in Table 3B not be used for these systems.

Recognizing the great potential offered by the proper use of precast and/or prestressed (particularly partially prestressed) components, and taking advantage of Sec. 1.5, it is proposed to modify Sec. 11.2 as follows.

MODIFICATION OF SEC. 11.2 Revise the second sentence to read: "Precast and/or prestressed reinforced concrete components may be used only if it can be shown by experiments and analysis based on established engineering principles that the resulting construction complies with the requirements of Secs. 3.6 and 3.7 and this chapter, and that they offer the same strength, stiffness, stability, durability, damping, and energy absorption and energy dissipation capacities (ductility) as required from the monolithic cast-in-place ordinarily reinforced concrete construction that they replace if the R and C_d values given in Table 3B are used."

1.2 SEC. 11.2 Revise capacity reduction factors for connections of precast components to read as follows:

Connection capacity as governed by concrete: ϕ=0.65
Connection capacity as governed by steel: ϕ=0.90

COMMENT: This proposed revision should not be introduced.

REASONS: The values suggested do not appear to be supported by reliable experimental data. The value of ϕ = 0.5 has been derived from the observed performance of connections of precast components during earthquakes and from analysis of data available from laboratory tests up to 1977. The observed earthquake performance of these connections either governed by concrete or by steel has been poor. Although it is recognized that, since 1977, new laboratory data have become available [Aswad, Spencer, Pall, Jurukovski (Yugoslavia) and others], these data are not sufficient to justify the proposed increase, particularly for the connection capacity as governed by steel, i.e., ϕ = 0.90. The argument given that Chapter 10 indicates a value of $\phi = 0.9$ for steel does not seem to be valid. Field inspections indicate that the quality control and workmanship used in the construction of joints in precast component connections are not the same as those of steel member connections. The uncertainties in design and construction in precast component connections seem considerably larger. Therefore, the ϕ cannot be the same.

1.3 SEC. 11.8 Revise the last sentence of paragraph 11.8.2 to read as follows: "Diaphragms for precast concrete floor systems may be developed with cast-in-place concrete topping, shear friction boundary reinforcing, or properly designed component connectors, either welded or grouted."

COMMENT: A modified revision is suggested.

REASONS: The proposed revision is not clear. It is not simply a question of constructing a diaphragm, but of developing sufficient resistance (strength), stiffness, stability, durability and ductility to guarantee the transmission of forces (inertia acting together with those due to gravity field and to other changes in environment conditions) to the seismic resisting system. Continuity should be assured in order to have stable resistance and stiffness under the combined stresses that can act in these diaphragms. A good technique to assure the satisfaction of these requirements appears to be the use of a cast-in-place topping. Recognizing that other techniques could be used in certain cases which could satisfy the above requirements, the following modified language could be developed (it should be noted that these are requirements for Categories C and D).

MODIFICATION OF SEC. 11.8.2 Revise the last paragraph of Sec. 11.8.2 to read as follows: "A cast-in-place topping on a precast floor system may serve as the diaphragm provided the cast-in-place topping is proportioned and detailed to resist the design shear forces under the effects of any loading combination (which could induce tensile or compressive stresses simultaneously to the shear forces). 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 ductility and stable hysteretic behavior, as the monolithic cast-in-place ordinarily reinforced concrete diaphragm, when subjected to cyclic loading conditions like those expected for seismic performance Categories C and D."

1.4 SEC. 11.8 Revise the first sentence of Section 11.8.4 to read as follows: "Boundary members shall be provided as required by Section 11.8.1 and 11.8.2, except for large panel precast concrete systems building construction with energy dissipating mechanisms formed in coupling links, as indicated in Section 11.11."

COMMENT: The proposed revision cannot be recommended.

REASONS: Lack of reliable experimental data regarding the behavior of large panel precast concrete systems building construction with energy dissipating mechanisms formed in coupling links, when subjected to severe earthquake ground shakings. The basis given for the proposed revision, i.e., reaseach at MIT and by Yugoslavs, Japanese and Russians, to the best of this writer's knowledge is far from adequate to justify the revision. The research conducted at MIT by Becker and Mueller is completely analytical, and they have pointed out clearly that, although the concept of coupling links seems quite promising, unfortunately there is very limited experimental data. Therefore, further development of such a concept requires additional analytical and experimental studies. Although the writer is familiar with some experimental work conducted by the Yugoslavs and Japanese on panel precast concrete systems buildings, the data available does not seem to justify the proposed change. It should be kept in mind that the requirement on the use of these boundary members is for the case of shear walls and diaphragms of seismic performance categories C and D, i.e., buildings which may be subjected to severe seismic motions and which therefore must be provided with large energy dissipation capacities. Note that the R values for these walls are 8 for tall buildings (dual system) and 5 1/2 for short buildings. Analysis of the experiments conducted by the PCA, in Berkeley and Japan, on R/C walls shows that, to obtain the ductility implied in these R values, it is necessary to have these boundary members. Again, the present ATC 3-06 does not prohibit the proposed use of coupling links (see Sec. 1.5), but substantiating evidence demonstrating that the proposed new system will have at least a seismic performance equal to the system recommended should be submitted.

1.5 SEC. 11.9 to 11.2 Add the following new sections to Chapter 11:

11.9 Plant Cast Prestressed Concrete
11.10 Post-Tensioned Concrete
11.11 Plant Cast Precast Concrete
11.12 Site Cast Precast Concrete

COMMENT: The writer supports the idea of the development of provisions for precast and prestressed concretes. These provisions should be grouped under a new subsection of Chapter 11. Precedents for doing so already exist. The recently proposed "Code of practice for the design of concrete structures" of New Zealand has a completely separate chapter of provisions for the design of prestressed and partially prestressed concrete members of fully ductile moment-resisting frames and joints between members. It should be pointed out, however, that in the development of these provisions it will be necessary to study the possibility of not only developing provisions peculiar to members with prestressing with the objective of developing the same strength, stiffness, stability, durability, and energy absorption and energy dissipation capacities (ductility) as the non-prestressed member, i.e., using the same R and C_d values, but also the possibility of assigning new values to R and C_d for precast and prestressed structures. At present perhaps all that can be done is to introduce the modification of Section 11.2 as proposed above in 1.1, noting that, in the case of prestressed members for seismic performance categories C and D, the prestressed members shall conform to the requirements of Sec. 3.7.12.

1.6 SEC. 11.2 Add at the end of the sentence reading "Axial compression or axial. . . . for the full height of the component.": "Non-lateral load resisting compression members designed in accordance with Sections 11.11 or 11.12 shall have capacity reduction factors as given in ACI 318-77."

COMMENT: The proposed revision cannot be recommended until provisions 11.11 and 11.12 have been developed.

REASONS: It should be noted that it will not usually be convenient to use a "pin-ended" compression member as a part of the lateral seismic force resisting system. If, in spite of this, such a pin-ended element is used as part of the seismic resisting system, because of the detrimental consequences of the interacting effects of the so-called nonstructural elements usually attached to the structural elements, it is believed a good policy to recommend the use of a reduced value of ϕ to discourage the use of elements without proper lateral reinforcement. A recent illustration of the need for special lateral reinforcement along the full height of the component is the failure of the ground-story columns of the Imperial County Services Building in El Centro.

1.7 SEC. 3.3.4 In Section (A) 3. - Delete the last sentence reading "This system is limited to buildings not over 240 feet in height."

COMMENT: This revision should be reviewed by Committee 2: Structural Design. It should be noted that, if this section is changed, Sec. 3.3.5 should also be changed.

1.8 SEC. 3.3.4 Add new type (A) 4: "Coupled shear wall systems with primary inelastic action along these vertical coupling elements providing energy dissipation."

COMMENT: This change should be discussed by Committee 2: Structural Design. The proposed system is not a new type, but the writer agrees with the basic idea put forward in the proposed revisions. As discussed and illustrated in several of his publications, the writer believes that the use of a structural system based on ductile walls coupled with girders having large energy dissipation capacity (large ductility and stable hysteretic behavior) leads to the best strong column-weak girder system, which is the basic requirement for the design of special (ductile) moment-resisting frames. Thus, the use of this system should be encouraged. Perhaps the best way to do so is in the commentary of Sec. 3.3.4.

1.9 SEC. 3.3.5 Delete the second sentence in this section in its entirety.

COMMENT: This is a subject for discussion and comments by Committee 2: Structural Design. Note that this height limitation is only for Category D and only for cantilever wall or braced frame systems. It does not apply to dual systems.

1.10 SECS. 7.4.4, 7.5.3 and 7.6.1

COMMENT: These proposed revisions should be reviewed by Committee 3: Foundations. The writer agrees with some of the statements made by Mr. Sheppard in his Basis, such as "Foundation requirements should be performance oriented" and in theory with the proposed revision to Sec. 7.5.3 and 7.6.1. However, it should be noted that the revision as stated is not complete. It will be necessary to specify a reliable method of analysis for the dynamic response of the soil-pile system. Furthermore, the proposed revision for these two sections cannot be accepted or even discussed until Section 11.9 is developed and accepted.

1.11 SECS. 8.2.2 and 8.2.3

COMMENT: These proposed revisions should be reviewed by Committee 8: Architectural, Mechanical and Electrical.

II. Nicholas Forell, SEAONC Representative

2.1 SEC. 11.8.1, 2 and 4 Need clarification that edge members or chords are required whenever tension stress exists in walls and diaphragms. These members must have ductile ties when computed gross section stress exceeds 0.2 f'_c, and only this requirement for ductile ties is discontinued when stress falls below 0.15 f'_c; but the edge member may be required for tension resistance beyond this level.

COMMENT: To introduce the requested clarification, the following changes are suggested:

Sec. 11.8.1 Add the following sentence at the end of the fifth paragraph of this section (top of page 108): "These shear walls shall have vertical boundary members along the edges as described in Sec. 11.8.4 at any level where tensile axial forces can be developed in the walls."

Sec. 11.8.2 Add the following sentence at the end of the second paragraph of this section: "Diaphragms shall have boundary members along their edges as described in Sec. 11.8.4 whenever tensile axial forces can be developed in these diaphragms."

2.2 SEC. 11.8.4 Boundary Members: Clarify last paragraph.

COMMENT: It is not clear what needs to be clarified.

III. Mark Fintel, PCA Representative

3.1 TABLE 1-B

COMMENT: These suggested changes should be reviewed by Committee 1: Seismic Risk Maps, and Committee 2: Structural Design.

3.2 SEC. 3.3.4 - page 46 Delete Section 3.3.4(A) [3]

COMMENT: This proposed revision should be reviewed by Committee 2: Structural Design. Although the writer favors the deletion of the limitation of 240 feet in height, he does not agree with the deletion of the rest of the other requirements recommended in 3.3.4(A) [3].

3.3 SEC. 3.9 - page 51 Add a new section 3.9.

COMMENT: This proposed addition should be reviewed by Committee 2: Structural Design. Although the writer favors the design of most of buildings using inelastic design procedures, and would like to be able to recommend a specific code method, based on inelastic design, which could be applied to all types of buildings, present knowledge does not permit this. Only in the case of very particular types of buildings to be built at certain sites in regions where sufficient seismic records exist is it possible to carry out rational inelastic design. One of the main problems is the sensitivity of the earthquake response of the building to (1) the dynamic characteristics of the ground motion, and (2) the modeling of the building (soil-foundation structural and nonstructural elements system) which should include the interacting effects of the nonstructural elements. Further study of these problems is required before a specific code inelastic design procedure can be recommended. At present the lack of reliable three-dimensional computer programs to analyze real buildings hampers advances in this field.

The main problems in the proposed "alternate inelastic design procedure" are, first, what constitutes "appropriate earthquake records" and, second, how is the "maximum rotational ductility required" defined and computed? The writer supports the idea of perhaps taking advantage of the following statement made in Sec. 3.1, "... with the procedures in Chapter 4 or Chapter 5 or an approved alternate procedure" to add, between parentheses, "The development and application of inelastic design procedures such as the one described in general terms in the commentary of this section is encouraged." If this suggestion is accepted, it is recommended that a special committee be appointed to study the method proposed by Mark Fintel.

3.4 TABLE 3-B - page 52 "Proposed Procedure to Develop Rational Values for Response Modification Coefficients, R"

COMMENT: The suggested procedure should be reviewed by Committee 2: Structural Design and other committees, since any change in this coefficient may affect other provisions.

The writer agrees with the need to review the R values. This need was expressed at the time the tentative provisions were formulated (see page 4 of the tentative provisions). Therefore, it is suggested that some effort be devoted to studying the reliability of the present recommended values of R. A committee should be appointed to conduct the required studies. The procedure suggested by Fintel might be considered as one possible procedure. In doing so, the comments made previously (3.3) should be kept in mind.

CRSI CONCRETE REINFORCING STEEL INSTITUTE

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JOSEPH G. MANNING, Regional Director – CRSI Western Region

January 31,1980

Members of Techncial Committee 4:
Concrete
Review of ATC 3-06

Victor Bertero	James Lefter
Edward Cohen	Richard Marshall
Mark Fintel	James Prendergast
Neil Hawkins	David Sheppard
Eugene Holland	

Gentlemen:

The following proposed revisions to ATC 3-06 are submitted for your consideration. These are written in code language, with appropriate reasons for each per instructions by the Committee Chairman.

Section 1.6.3(A) Special Testing

Revise exception under paragraph one as follows:

Exception:

Certified mill test certificates may be accepted for ASTM A-706 and A-615 reinforcing steel.

Reason: Section 11.5.1, as suggested to be revised, would permit ASTM A615 grade 40 or 60 reinforcement. Mill tests specify actual yield and tensile strengths.

Table 1-B Seismicity Index

Revise the "Seismicity Index" of table 1-B to read as follows:

Map Area Number	Seismicity Index
7	4
6	4
5	~~4~~ 3
4	~~3~~ 2
3	2
2	~~2~~ 1
1	1

Reason: The seismicity index relates to various levels of detailing requirements through the seismic performance category. Under ATC 3-06, in some areas such as Phoenix,AZ.,detailing requirements would be increased to the same level as that for say San Francisco. This obviously should not be required. It is therefore recommended to maintain current levels of detailing that have been determined by the local engineering profession.

Section 3.3.5 Seismic Performance Category D

Delete second paragraph modifying height limitations of Sec.3.3.4

Reason: These limitations are considered overly restrictive and arbitrary. Height alone is no critera for the performance of a structure under seismic loading. The limitations in Section 3.3.4 would appear to provide adequate performance.

Table 3-B Response Modification Coefficients

Revise table 3-B as follows:

Type of Structural System	Coefficients R	C_d
Bearing Wall System: ...	4	4
Building Frame System: ...	6	4
Moment Resisting Frame System:...	8	6
Dual System: ...	7	5
Inverted Pendulum Structures: ...	2½	2½

Reason: The Response Modification Factors,R, are out of necessity arbitary. These numbers will have a significantly greater impact on the construction industry than the current K values because of the more detailed breakdown of systems and materials. It is obvious that these values must be determined by a rational means and not arbitrarily selected. Until such time as this can be done it is suggested that the R coefficients for the type of structural system be selected (similar to the method used for the current K values) rather than R values for individual systems and materials.

Section 4.2.2 Period Determination

Revise equation (4-4) as follows:

$$T_a = 0.10\ N$$

where N = The total number of stories above the base to the highest level of the building.

Reason: This is recommended for two reasons. First, for simplicity. Second, the coefficient C_T affects the base shear out of proportion to its significance. For example the period for a 15 story frame affects the base shear twice as much as the response modification factor. This great great of an impact on the base shear is not warrented.

Section 7.5.3(C) Precast Concrete Piles

Revise second sentence to read as follows:

> Precast concrete and prestressed precast concrete piling shall be designed to withstand maximum imposed curvatures resulting from the maximum soil deformations that would occur during an earthquake.

Reason: Prestressed precast concrete piling can withstand considerable curvature and through proper detailing confinement and ductility can be provided.

Section 7.6.1 Special Pile Limitations

Delete this section.

Reason: See comments on Section 7.5.3 .

Section 11.1 Reference Documents

Revise reference 11.1 as follows:

> Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute. (ACI 318-77 but excluding Appendix A)

Reason: Reference should be to the most recent edition of ACI standard. References to sections throughout document must be revised as appropriate if this change is approved.

Section 11.2 Strength of Members and Connections

Revise third paragraph as follows:

> Axial compression or axial compression combined with bending on any member where axial stress due to all loads exceeds $0.10f'_c$ and the axial stress due to seismic forces exceeds $0.05f'_c$ and special lateral reinforcement as specified in section 11.7.2 (C).

Reason: The statement for the full height of the component is misleading. Further, the extent of special lateral reinforcement is specified in Section 11.7.2 (C).

Section 11.5.1 Material Requirements

Revise third paragraph, second sentence as follows:

> ASTM A-615,grade 40 <u>or 60</u> reinforcement may be used in these elements if . . . .

Reason: ASTM A615 grade 60 will meet the physical requirements desired and it has performed satidfactorily in its current use. Further A706,which is also a grade 60 reinforcement is not readily available at the present time.

Section 11.8.1 Shear Wall Details and Limitations

Delete third paragraph reuiring two curtains of steel.

Reason: This provision seems overly restrictive. Many tilt-up buildings with a single curtain of steel have performed satisfactorily under earthquake loads. Further, any shear wall in a major structure and one carrying high shear is likely to be 10 inches or more in thickness. Section 14.2(g) of ACI 318-71 requires 2 curtains of reinforcement in such walls.

Time for review does not permit a more detailed study of this document. Therefore additional comments may be forthcoming later in the review process.

Sincerely,

Joseph G. Manning
Regional Director

JGM:jm

cc: Dr. Gene Corley
    Paul Rice

Received @ S.F. [illegible]
Feb. 21, 1980 [illegible]

February 11, 1980

COMMENTS ON PROPOSED REVISIONS TO ATC 3-06

V. V. Bertero, Representative of ATC

REVISIONS PROPOSED BY:

IV. Joseph G. Manning, Representative of the Concrete Reinforcing Steel Institute

4.1 SEC. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-77 but excluding Appendix A)

COMMENT: This revision should be introduced but modified as follows: "Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-77) excluding Appendix A and replacing Section 9.2.3 by Section 3.7.1 of this document," or the following alternative: ". . . provisions of this ATC whole document, Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute (ACI 318-77) excluding Appendix A."

REASONS: The purpose of excluding Section 9.2.3 is to remind the designer that the combination of load effects adopted by ATC 3-06 differs from that required by ACI 318-77. For this reason I prefer the first alternative revision. I had already proposed this correction together with these related revisions.

SEC. 11.2 3rd paragraph should read: "The capacity reduction factor in Ref. 11.1, Sec. 9.3 shall be modified as follows:"

SEC. 11.7.1 5th paragraph, second line should read: ". . . splicing conforming to Ref. 11.1, Sec. 12.15.3 may be used."

SEC. 11.7.2(B) 1st line at top of page 106 should read: "Welded splices and approved mechanical connections conforming to Ref. 11.1, Sec. 12.5.3 may be used . . ."

Appropriate revisions should be made also in the Commentary of Chapter 11, i.e., pages 449, 450, 451, and 453.

4.2 SEC. 11.2 Revise third paragraph as follows: "Axial compression or axial compression combined with bending on any member where axial stress due to all loads exceeds 0.10 f'_c and the axial stress due to seismic forces exceeds 0.05 f'_c and special lateral reinforcement as specified in Section 11.7.2(C)."

COMMENT: The proposed revision should be introduced.

4.3 SEC. 11.5.1 Revise third paragraph, second sentence, as follows: ASTM A-615, grade 40 or 60 reinforcement may be used in these elements if . . .

COMMENT: This revision could be introduced if, at the end of the paragraph, the following addition is made: ". . . and (3) the welding, when it is required, can be shown to offer the same strength and toughness as the ASTM A-706 under cyclic loading including strain reversals."

4.4 SEC. 11.8.1 Delete third paragraph requiring two curtains of steel.

COMMENT: This revision should not be introduced. Walls are subjected to bending about their weak axis due to inertia forces perpendicular to the plane of the wall. There are cases, particularly when barbell cross-section walls are used, when the thickness of the wall panel could be less than 10 inches.

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URGENT

Dr. Richard D. Marshall, Secretary
Technical Committee 4: Concrete
Review and Refinement of ATC 306
U.S. Department of Commerce
National Bureau of Standards
Washington, D.C. 20254

February 11, 1980

Dear Dick:

With respect to Chapter 11 - Reinforced Concrete, it appears that this is based on an early draft of the pending revision of ACI 318, Appendix A. This draft has been substantially revised since it was used to develop Chapter 11. I have been assured that a revised, nearly final, draft will be available for our Committee meeting on February 21, and that the final draft will be available for the Trial Design phase of ATC-306.

This draft is scheduled to go to the full Committee 318 on March 1 at its meeting in Las Vegas and can be balloted by Committee 318 immediately thereafter.

It would then be available for publication and membership ballot in January 1981. Future revisions of Appendix A could be readily coordinated with the appropriate committees of ATC, which have overlapping membership with ACI 318, and published concurrently with future ATC 306 revisions.

Under these circumstances and since ACI 318 including Appendix A is a fully approved concensus document, to avoid overlapping and conflicting efforts and criteria, and to assure that Chapter 11 represents the highest practical state of the art, I strongly recommend that, in the national interest, Reference 11.1 read as follows:

"Building Code Requirements for Reinforced Concrete, American Concrete Institute. (ACI 318 Current Edition)."

I apologize for the delay in forwarding this recommendation but trust that you will be able to distribute it to all members of Committee 4 and other appropriate groups.

Very truly yours,

Edward Cohen

EC:mrm

UNIVERSITY OF WASHINGTON
SEATTLE, WASHINGTON 98195

Department of Civil Engineering

February 26, 1980

Dr. R. D. Marshall
Secretary, Technical Committee 4
ATC 3-06 Review
U. S. Department of Commerce
National Bureau of Standards
Washington D. C. 20254

Dear Dick:

As promised February 21, 1980, I enclose my suggested revisions to ATC 3-06 for the tasks assigned to me at our December 11 meeting in Washington D. C. These comments assume that the committee is already proposing to amend ATC 3-06 as agreed at our San Francisco meeting. My suggested revisions therefore address only two substantive issues:

(1) Section 3.7.12. Vertical Seismic Motions for Buildings Assigned to Categories C and D, and

(2) Seismic Provisions for Flat Plate, Flat Slab, and Waffle Slab Structures.

My input for the latter issue consists of three parts:

(1) A discussion of the apparent implications of ATC 3-06 if unamended for flat plate construction.

(2) A review of the state of knowledge on the behavior of flat plate construction under reversed cyclic loading.

(3) A suggested set of revisions to the provisions and the commentary, and a set of reason statements for the revisions.

My discussion uses the term flat plate to cover flat slab and waffle slab structures as well as flat plate structures in both reinforced and prestressed concrete.

Sincerely,

Neil M. Hawkins

Neil M. Hawkins
Professor and Chairman
PTI Representative
sr
cc: Members AT 3-06, Technical Review Committee 4
W. G. Corley, M. A. Sozen, J. Grossman, C. L. Freyermuth (with enclosures)

(1) Section 3.7.12 Vertical Seismic Motions for Buildings Assigned to Categories C and D

The vertical component of earthquake motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. For horizontal cantilever components, these effects may be satisfied by designing for a net upward force of $0.2Q_D$. For horizontal prestressed components, these effects may be satisfied by designing for a net upward force of $0.2Q_D$ if the prestressed component is not part of the seismic resisting system and bonded reinforcement not less than the minimums required by these provisions is provided where maximum moments may occur for both horizontal and vertical components of the earthquake motion. For other horizontal prestressed components, these effects may be satisfied by Formula 3-2a.

REASON

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 brittles failures do not occur and seismic provisions can be consistent with those for reinforced concrete units.

(2) FLAT PLATE CONSTRUCTION

1. General

1.1 Exposure Group B

For buildings of seismic hazard exposure group B, ATC 3-06 effectively prohibits flat plate construction. Group B includes all facilities in New York, Boston, Buffalo, and the New England states, much of North and South Carolina and Tennessee, large areas of Oklahoma, Arkansas, Illinois, and Missouri, and much of New Mexico, Montana, Idaho, Utah, Washington and Wyoming. The potential economic impact of that prohibition is staggering. For the post-tensioning industry alone, that prohibition could mean about a 25% drop in its work volume. Post-tensioning is used in about 30 million square feet of suspended slabs constructed each year in the U.S.A. and much of that construction is flat plate.

In his letter of November 19, 1979, to ATC, Jacob Grossman of Robert Rosenwasser and Associates of New York, details his experience with respect to the economics of flat plate construction and its seismic response when properly detailed. He states "I cannot even begin to describe the construction havoc the exclusion of flat-slab structures can introduce in strong union-high construction cost areas." He points out that enough research and knowledge are available to incorporate flat-slabs and allow them as "ordinary" frames without shear reinforcing.

It is not clear that it was intended that ATC 3-06 prohibit flat-slab construction for "ordinary" frames. Contrary to the situation for group C and D exposures, neither the provisions nor the commentary explicitly state such a prohibition. However, they require in flexural members of ordinary frames for group B exposure, web reinforcement perpendicular to the longitudinal reinforcement throughout the length of the member. The minimum reinforcement is two leg No. 3 stirrups at a spacing of d/2. Inclusion of such reinforcement in flat slabs would create economic as well as logistic problems.

It is suggested that:

(1) the provisions state clearly whether flat plate, flat slab, or waffle slab construction is feasible for ordinary frames for Category B.

(2) If flat plate, flat slab, or waffle slab construction is feasible, the provisions specify any special restrictions for that construction.

1.2 Exposure Groups C and D

The intention of ATC 3 with respect to restrictions on the use of flat plate framing to resist lateral forces for category C and D structures is clear. Such use is highly undesirable. However, the intention for category C and D structures where seismic forces are resisted by a parallel structural system, is not clear. In the Commentary to Chapter 11 on page 450 dealing with Material Requirements for Seismic Categories C and D it is stated:

"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."

That statement is logical and non-controversial. However, the statement continues,

"Therefore, all frame components (which are not designed to resist seismic forces) in Categories C and D buildings, are required to have as a minimum, the details specified in Sec. 11.6."

That section gives requirements for ordinary moment frames assigned to Category B.

"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 resistance, those components must have special transverse reinforcement as specified for Special Moment Frames."

Effectively, those statements prohibit the use of any flat plate frame, slab, or waffle slab in Category C and D structures. They require special transverse reinforcement of a type illogical for flat plates, especially if yielding is predicted. How to define yielding in a flat plate is also unclear.

The Commentary for Section 3.6.3 on Seismic Performance Category C is more explicit on page 346. It states:

"In many buildings, the seismic resisting system does not include all of the components that support the gravity loads. A common example would be a flat slab concrete warehouse of several stories in height where the lateral seismic loads are resisted by exterior shear walls or exterior ductile moment resistant frames..."

"Subsec. (c), (apparently refers to Section 3.3.4(c)), requires that the vertical load carrying capacity be reviewed at the actual deformations resulting from the earthquake. In the example of the flat slab warehouse, there will be bending moments in the columns and slabs and an uneven shear distribution at the column capitals. At the calculated deflections and the resulting imposed moments and shears, it must be demonstrated that the members and connections will not fail under the design gravity loadings."

That statement is logical and non-controversial. However, the statement continues:

"The loading is cyclical so static ultimate load capacities are not acceptable. If the combination of those loads and deformations result in stresses below yield, it can be assumed that the system is capable of supporting the gravity loads. If the stresses are above yield, then sufficient ductility under cyclic loading must be provided. If the [illegible] load bearin[illegible] system is to provide any calculated resistance to seismic resistance (no matter how small), then the detailing for ductility must be consistent with the values given in Table 3-B."

What yielding is for a slab-column connection is not defined and for the example discussed what ultimate load capacity would be acceptable for cyclic loading is not defined in ATC 3-06 or ACI 318-77.

Finally, the Commentary on Section 11.7.1 Flexural Members in Special Moment Frames on page 453 states:

"The geometric constraints given for flexural members are based primarily on past practice. The maximum width limitation explicitly and intentionally eliminates the use of a flat plate or flat slab working as a frame unless special details are incorporated in the structure. It should be pointed out that even if it may be possible to provide the necessary flexural strength in that portion of the slab permitted to be designated as a beam, it is likely that the drift criteria will govern the design for Categories C and D. Furthermore, if a flat plate or a flat slab is used as a frame working parallel with a structural wall, the actual relative stiffnesses of these two systems in the non-linear range of response should be evaluated elastically considering the effect of cracking and reinforcement slip, rather than on the basis of gross section."

That statement is correct and consistent with test data.

2. Behavior of Flat Plate Construction under Cyclic Loading

2.1 Laboratory Results

There have been seven major laboratory investigations of flat plate connections subjected to cyclic loading (1-8). The results of the extensive University of Washington investigations are summarized in the attached articles.

The lateral load response is strongly influenced by:

(1) the amount and distribution of the flexural reinforcement in the slab,

(2) by the amount, type, and extent of any shear reinforcement, and

(3) by the level of shear stress transferred to the column simultaneous with the moment.

Even when there has been a low flexural reinforcement ratio and a connection well over-designed for shear, there has still been little ductility under reversed cyclic loading. For specimens with high reinforcement ratios within lines one and one-half times the slab thickness either side of the column, there has been a considerable increase in the lateral load stiffness. However, there has been as much as a 10% reduction in the moment transfer capacity as compared to that for monotonic loading and a punching failure has occurred shortly after the reinforcement passing through the column has yielded. Since there is little improvement in the ductility with the use of low reinforcement ratios and a considerable reudction in stiffness, concentration of column strip reinforcement is desirable.

The only proven ways of maintaining capacity through large rotations has been to add properly detailed shear reinforcement consisting of either integral beam stirrups or thin steel H sections or studs anchored above and below the flexural reinforcement passing through the column. Shear reinforcement in the form of shearheads or bent bars increases the capacity but does not increase ductility. The shear reinforcement must hold the top and bottom flexural mats together and prevent the development of a splitting crack between those mats. The shear reinforcement should have a spacing not exceeding d/2 and need not extend further than about 5 slab thicknesses out from each column face. Rules for proper detailing of such shear reinforcement are described in Reference 7. Slab-column connections are so flexible that flat plate structures are unlikely to meet ATC 3-06's stiffness requirements for ductile moment resistant frames. Thus, shear reinforcement in flat plates is probably unnecessary unless the flat plate structure is the only line of defense or unless the flat-plate structure is to provide a required secondary line of defense.

The level of shear stress transferred simultaneously with the moment markedly affects the energy dissipation and ductility characteristics of slab-column connections. To obtain desirable characteristics, the flexural reinforcement within lines one and one-half times the slab thicknesses, either side of the column should be limited to one percent and the shear stress due to shear transfer on the critical section d/2 from the column perimeter to $2\sqrt{f_c'}$. At that latter stress, shear cracks have not developed in the slab prior to the application of lateral loading.

After a punching failure has occurred, bottom bar flexural reinforcement continuous through the column is essential to the connection being able to maintain its gravity load carrying capacity. Such reinforcement can carry a shear force equal to its shearing yield capacity. Alternatively, prestressing reinforcement passing through a column or over a lift slab collar is also a very effective means of tying a slab-column connection together after a punching failure. With prestressing reinforcement, a residual capacity can be obtained equal to 90 percent of the pre-punching shear capacity.

Shear or torsional cracking develops at the discontinuous edge of a slab adjacent to an exterior column, when the shear stress at that location evaluated according to ACI 318-77 provisions, exceeds $2\sqrt{f_c'}$. If that stress is exceeded, stirrups having a size not less than No. 3, a spacing equal to or less than d/2, and extending up to four times the slab thickness from the torsional faces of the column should be provided to prevent opening of those cracks. While the best ductility and energy dissipation characteristics are obtained with integral beam stirrups, hairpin stirrups inserted perpendicular to the edge and extending a distance equal to the column projection into the slab plus ℓ_d, or twice the slab thickness plus ℓ_d, whichever is less, into the slab will also provide adequate control to the opening of those cracks.

Tests have shown that the above results are also applicable to waffle slab-interior column connections (4) and that when there is moment transfer about both axes of a column (8), the effect of the minor moment on the shear capacity can be neglected if that moment does not exceed 30% of the major moment and there is adequate reinforcement within lines one and one-half times the slab thickness either side of the column to transfer that minor moment.

The recent tests on flat plate frames at the University of Washington (8), have shown that in a frame, a punching failure will occur at a connection without stirrups at a displacement and at a capacity consistent with subassemblage results. However, that punching failure did not lead to an immediate loss in capacity of the overall frame since the adjacent connection with shear reinforcement was able to supply the required additional moment transfer capacity. Displacements much greater than those for punching at the connection without shear reinforcement had to be applied before the capacity of the overall frame deteriorated. During that period the connection without shear reinforcement continued to carry its share of the gravity load. Further, after lateral loading was completed, it was found that at the punched connection the slab could be readily jacked back up to its original elevation and the connection repaired.

2.2 Field Results

Reports (9) have been issued on the behavior of several flat plate structures in the San Fernando earthquake; the Holiday Inn, Orion Avenue, the Holiday Inn, Marengo, and the Muir Medical Center. The Holiday Inn, Orion Avenue, was a seven-story reinforced concrete flat plate structure with typical plan dimensions 62 by 160 ft. The building was supported on piles centered under the columns which were spaced at approximately 20 ft. centers. Spandrel beams approximately 16 x 22-1/2 in. surrounded the perimeter of the structure. The flat plate floor was 10 in. thick at the second floor, 8 in. thick at the roof, and 8-1/2 in. thick for all other floors. The spandrel beams were figured as creating exterior frames roughly twice as stiff as the interior flat plates so that in the short and long directions of the building, 36% and 67%, respectively, of the stiffness was provided by the exterior frames. Peak accelerations at the first floor level were 0.25g and 0.13g in the short and long directions, respectively. Roof accelerations were 0.41 and 0.33g, respectively. Repairs cost 11% of the initial construction cost and were nearly all non-structural. Some structural distress occurred at the corner column beam connections and in the construction joints at the soffit of the exterior column-beam connections. The response was most marked in the short direction with a lengthening of the period part way through the record indicating that the structure began responding inelastically. The analysis indicated that beams and slabs yielded, that columns generally remained elastic, and that interstory drifts as large as 0.13 ft. occurred. The elastic limit displacement was roughly 2-1/2 times the design code displacement.

The Holiday Inn, Marengo Street, had dimensions and member sizes almost exactly the same as the Orion Avenue building. Peak accelerations at the first floor level were 0.15g and 0.14g in the short and long directions, respectively. Roof accelerations were 0.43g and 0.25g, respectively. Repairs cost 7% of the initial construction cost and were nearly all non-structural. Structural distress was similar to that for the Orion Avenue building. The dynamic response was also similar to the Orion Avenue building. The analysis indicated that beams and slabs yielded at their connections with the columns but that the columns generally remained elastic. Interstory drifts were as large as 0.14 ft. In this structure, as in the Orion Avenue structure, it was apparent that the stiffness of the frame was sufficiently low that the non-structural elements such as partitions, played an appreciable role in the character of the structural response to seismic forces.

The Muir Medical Center was an 11-story office tower with an 18-1/2 in. deep waffle slab at the ground level and perimeter basement walls. For the second floor and above, the 9-in. thick flat slab had 15 in. deep tapered column capitals with deep spandrel beams around the perimeter. The deep spandrel beams framing provided 70% of the lateral load stiffness and the interior flat-slab-tapered drop panel framing the other 30%. Peak accelerations were 0.10g at the basement level and 0.2g at the roof level. Some of the structural members were predicted to yield during the earthquake and the maximum story drift was computed to be 0.64 in. The general performance of the structure was linear-elastic with only minor lengthening of the building period during the earthquake. Damage was all non-structural and estimated at less than $2,000.

2.3 Period of Vibration

Fundamental periods for those structures for man-induced excitations prior to the earthquake, at the beginning of the earthquake, mid-way through the earthquake, and for man-induced excitations after the earthquake are listed in Table 1. It is apparent that the period of the predominantly flat plate structures, the Holiday Inns, increased noticeably during the earthquakes with the increase being larger for the more heavily shaken Orion Avenue building.

TABLE 1
Periods of Vibration for Flat Slab Structures

			Periods			
	Stories	Direction	Before Quake	Start of Quake	Mid-Way Through Record	After Quake
Holiday Inn Orion Avenue	7	Short	0.48	0.79	1.6	0.68
		Long	0.53	0.88	1.24	0.72
Holiday Inn Marengo	7	Long	0.53	0.88	1.0	0.64
		Short	0.49	0.79	1.2	0.63
Muir Medical	11	Long	0.90	1.43	1.4	1.02
		Short	1.03	1.60	1.6	1.14

If the values of the periods at the start of the earthquake are compared with the general data on Fig. C4-2, page 373 of ATC 3-06, then it is apparent that the periods for these structures are better characterized by the relationship, $T_R = 0.035\ h_n^{3/4}$ than the relationship $T_R = 0.025\ h_n^{3/4}$.

2.4 Stiffness

The ATC 3-06 provisions limit the allowable story drift for Seismic Hazards Exposure Groups I and II to 0.015 radians. When there are no brittle-type finishes in buildings three stories or less in height, those values can be increased to 0.02 radians. If it is accepted that for a reinforced concrete structure, a load factor of 1.4 is required on earthquake forces and a capacity reduction factor of 0.9 for flexure, then connection rotations at 65% of the ultimate capacity should not exceed the story drift specified above divided by C_d. Maximum measured values of C_d for a story drift of say 0.015 radians for a given column proportion and spacing can be evaluated directly from the subassemblage specimens.

Table 2 lists C_d values calculated according to that concept for several different investigations. Values range from a low of 2.4 for the flat plate frame test (8) with a low ρ value through to a high of 4.3 for the waffle slab specimen (4). There is a marked increase in values with increasing slab depth and a lesser increase with increasing column size. Not apparent from that table is the wide variation in results obtained for supposedly identical specimens. C_d values varied by as much as 50% for similar specimens and averaged about 20% higher for specimens with shear reinforcement than those without.

Based on these subassemblage results and experience from the San Fernando earthquake, it is apparent that a conservative value of C_d for flat plate structures is 2. Although higher values can be obtained by careful detailing, even for waffle slabs, it is unrealistic to expect that the C_d value of 6 required for a ductile moment resistant frame can be obtained. Thus, flat plate framing should only be recognized as an acceptable lateral load resisting system when classified as an ordinary frame. The only possible exception might be for a waffle slab structure without brittle finishes and less than three stories high. Even in that case, experience from the San Fernando earthquake with the Olive View Hospital Ambulance Port was undesirable. However, the 14 x 18 in. columns were smaller than desirable and failure occurred in the columns and not in the slab-column connection.

2.5 Conclusions

Based on this summary of field and laboratory experience, it is concluded that:

(1) flat-plate structures of normal proportions and without shear reinforcement will have little difficulty in meeting the strength, stiffness, ductility, etc., requirements for ordinary frames, especially if certain detailing requirements specified later, are satisfied.

TABLE 2

VALUES OF C_d MEASURED IN SUB-ASSEMBLAGE TESTS

Reference	Scale	Full Scale Properties				C_d	Specimen Type
		Slab Thickness in.	Column Spacing ft.	Column Size in. x in.	Story Height ft.		
1	3/8	8	18	16 x 16	13	2.5	Flat Plate
2	Full	7.5	19	18 x 18	11	2.5	Flat Plate
3	0.4	10	20	20 x 20	10	3.5	Flat Plate
4	1/4	14	20	20 x 20	11	4.3	Waffle Slab
7	3/4	8	16	21 x 21	11	2.8	Flat Plate
8	1/2	9	24	19½ x 8	9	2.4	Flat Plate Frame-loop

(2) With flat plate structures of normal proportions it would be extremely difficult, if not impossible, to meet the stiffness requirements for utilizing such frames as special moment frames for Category C and D buildings.

(3) With flat plates of normal proportions punching failures will not occur until interstory drifts greater than the limiting values specified in Table 3-C, page 53, of ATC 3-06.

(4) With flat plate structures used as the gravity load carrying system in Category C and D buildings, it is not necessary to consider punching failures as unacceptable provided the detailing requirements, specified later, are satisfied.

(5) With flat plate structures yielding should be defined as either:

(a) the development of the negative moment yield capacity of the slab on a line extending across the width of the slab at the column face, or

(b) the development of the moment transfer capacity at the slab-column connection for yielding of the reinforcement at that connection. That capacity can be taken as the flexural capacity of the reinforcement top and bottom within lines one and one-half times the slab thickness either side of the column.

(6) The period of structures with 35% or more of the lateral load stiffness provided by flat plate framing can be estimated from the relationship $T_R = 0.035\ h^{3/4}$

2.6 REFERENCES

1. Hanson, N. and Hanson, J., "Shear and Moment Transfer Between Concrete Slabs and Columns," Journal of the PCA Research and Development Laboratories, January 1968. $C_d = 2.5$

2. Carpenter, J.E., Kaar, P.H. and Corley, W.G., "Design of Ductile Flat Plate Structures to Resist Earthquakes," Proc. 5th World Conf. Earthquake Eng., Rome, Italy, 1973.

3. Kanoh, Y. and Yoshizaki, S., "Experiments on Slab-to-Column and Slab-to-Wall Connections," Japan Concrete Journal, Vol. 13, No. 6, June 1975 $C_d = 3.5$

4. Rodriguez, M.R., "Diseno Sismico De Conexiones Entre Losas Planas Reticulares y Columnas," M. E. Thesis, University of Mexico, July 1979. $C_d = 4.3$

5. Islam, S. and Park, R., "Tests on Slab-Column Connections with Shear and Unbalanced Flexure," Journal of the Structural Division, ASCE, Vol. 102, No. ST3, March 1976, pp. 549-568.

6. Zaghlool, E.E.R., "Strength and Behavior of Corner and Edge Column-Slab Connections in Reinforced Concrete Flat Plates," Ph.D. Thesis, University of Calgary, Alberta, Canada, 1971.

7. 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.

8. Hawkins, N.M., "Seismic Response of Concrete Flat Plate Structures," Proc. Seventh World Conférence on Earthquake Engrg., Istanbul, 1980.

9. San Fernando, California, Earthquake of February 9, 1971, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1973.

3. Suggested Revisions

3.1 Page 103, 11.6.1, Flexural Members

A. Alter sentence 2 of paragraph 2 as follows:

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

B. Alter sentence 1 of paragraph 6 as follows:

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

C. Alter sentence 1 of paragraph 7 as follows:

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

D. Add paragraph 8 as follows:

"Slabs without beams and supported on columns may be used for ordinary moment frames provided those slabs satisfy the requirements of Chapter 13 of Ref. 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:

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

where V is the shear force transferred to column due to unfactored gravity loads and V_p 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 as torsion reinforcement. 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.

Reasons

Section 11.6.1 is altered so that is clear that flat plate, flat slab, or waffle slab construction is acceptable for ordinary frames. Revision D spells out special restrictions for that framing.

A, B, and C. Elimination of requirements for slabs is viable when Revision D is added.

D. Sentence 1 requires that slabs be defined according to the two-way systems envisaged in Chapter 13 of ACI 318-77.

Sentence 2 requires sufficient reinforcement through a column to be able to support the gravity load of a slab in the unexpected event that a punching failure occurs.

Sentence 3 specifies a minimum amount for that reinforcement.

Sentence 4 creates a situation where the steel passing through the column head area will yield shortly before or simultaneously with yielding on a maximum negative moment line extending across the width of the slab. If that condition is not satisfied and only the requirements of Chapter 13 of ACI 318-77 satisfied the negative moment reinforcement passing through the column can be yielding under gravity loads. The lateral load stiffness of the flat plate framing would be decreased markedly.

Sentence 5 requires that the shear stress caused by the gravity loads will be sufficiently low that the connection will have a ductility ratio of at least 2.

Sentences 6 and 7 add requirements identified in the Suggested Revisions to ACI Code 318-77 by ACI-ASCE Committee 426 and the previous discussion. Sentence 6 ensures that if shear or torsional cracks develop at the edges of slabs, there is reinforcement that can control the opening of those cracks. Sentence 7 specifies how to distribute shears when there is a spandrel beam and no flexural beam.

3.2 E. Page 451 in Commentary

Revision

Add to end of paragraph for 11.5.1 finishing on that page:

"Flat plate frames of normal proportions and detailed as specified in Sec. 11.6 will not yield until story drifts greater than 0.03 h_{sx}."

Reason

Clarifies that flat plate frames are very flexible relative to other framing systems and corrects deficiency noted in discussion.

3.3 F Page 347 of Commentary

Revision

Alter starting at the top of page 347 as follows:

"Loading is cyclical, so static ultimate load capacities may not be reached. If ... (to end of paragraph). In the example of the flat plate warehouse, the connections can still carry the design gravity loadings if they satisfy the requirements of Sec. 11.6.1. If, however, they are to provide any calculated seismic resistance, they must be detailed for ductility consistent with that required for the parallel exterior shear wall or ductile moment resisting frames."

Reason

Clarification of wording to make it consistent with the revised Sec. 11.6.1.

3.4 G Section 4.2.2 Page 56

Revision

Alter Eq. (4-4) as follows:

"where C_T = 0.035 for steel frames or concrete frames where flat plate framing provides 35% or more of the lateral load stiffness."

Reason

Flat plate framing is considerably more flexible that beam and column framing, slab sections are lightly reinforced, and cracked at the column face under gravity loadings. The three structures shown on Fig. C4-2 Page 373 satisfying the 35% requirement had initial periods at the start of the earthquake on or above the broken line in that figure and their periods increased with the duration of shaking. Part of the greater initial stiffness was attributed in the San Fernando reports to the stiffening effects of non-structural elements. Closer attention to architectural requirements, as specified in Chapter 8, would increase the conservancy of calculating C_T values as recommended above.

3.5 H Page 348 of Commentary

Revision

Add to last paragraph of Page 348:

"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."

Reason

Considerable simplification that is predictable using beam-analogy concepts (7, 8) and has been proven by testing (8).

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March 25, 1980

TO: Members of Technical Committee 4: Concrete

Edward Cohen
Eugene Holland
Joe Manning
Jim Prendergast
Mark Fintel
Neil Hawkins
Vitelmo Bertero
Jim Lefter
Dick Marshall

RE: ATC 3-06 Review

Gentlemen:

As discussed and committed in our 21 Feb 80 meeting, I have developed recommended detailing requirements for prestressed concrete piling, to be inserted into Chapter 7.

Add new Section 7.5.3 (E)

(1) For the body of fully embedded foundation piling subjected to vertical loads only, or where the design bending moment does not exceed 0.2 $\emptyset$ M_{NB}, 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:

$$S_{sp} = \frac{f_y A_{sp}}{(c + 7\, d_{sp})\, f_r}$$

where S_{sp} = spacing of spiral reinforcing

f_y = yield strength of spiral reinforcing

A_{sp} = Area of spiral reinforcing

c = cover over the spiral reinforcing

dsp = diameter of spiral reinforcing

fr = modulus of rupture of concrete

ρ_s = ratio of volume of spiral reinforcing to total volume of core (out to out of spirals)

(3) Any piling installed in layered soils imposing severe curveatures 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 - curveature 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.

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.

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 Co.

Very truly yours,

David A. Sheppard

David A. Sheppard
California Marketing Director

cc: California Steering Committee
Dan Jenny, PCI
Ben C. Gerwick, Jr.

Received @ PCA April 14, 1980

April 7, 1980

PROPOSED REVISIONS TO ATC 3-06

V. V. Bertero - Representative of ATC

REVISIONS PROPOSED BY:

V. Neil M. Hawkins, PTI Representative

5.1 SEC. 11.6.1 Flexural Members, page 103

A. Alter sentence 2 of paragraph 2 as follows:

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

B. Alter sentence 1 of paragraph 6 as follows:

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

C. Alter sentence 1 of paragraph 7 as follows:

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

D. Add paragraph 8 as follows:

"Slabs without beams and supported on columns may be used for ordinary moment frames provided those slabs satisfy the requirements of Chapter 13 of Ref. 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:

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

where V is the shear force transferred to column due to unfactored gravity loads and V_p 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 as torsion reinforcement. 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.

COMMENT:

Revision A should be introduced.

Revision B should be introduced.

Revision C should be introduced.

Revision D should be introduced but with the following correction and addition. The end of sentence 6 of this suggested new paragraph 8 should read as follows: ". . . it can act effectively as torsion reinforcement considering the possibility of full reversals of the sense of the torsional moments."

Furthermore, it is suggested that in the commentary to this Section 11.6.1 some guidelines regarding the proper detailing of such torsion reinforcement be given.

5.2 COMMENTARY ON SEC. 11.5.1 on page 451.

Add to end of paragraph for 11.5.1 finishing on that page: "Flat plate frames of normal proportions and detailed as specified in Sec. 11.6 will not yield until story drifts greater than 0.03 h_{sx}."

COMMENT: This proposed revision should be introduced modified as follows: "The flat plates of flat plate frames of normal proportions and detailed as specified in Sec. 11.6 will not undergo any significant yield until story drifts greater than those allowable (Table 3-C)."

5.3 COMMENTARY ON SEC. 3.6.3 on page 347.

Alter starting at the top of page 347 as follows:

"Loading is cyclical, so static ultimate load capacities may not be reached. If . . . (to end of paragraph). In the example of the flat plate warehouse, the connections can still carry the design gravity loadings if they satisfy the requirements of Sec. 11.6.1. If, however, they are to provide any calculated seismic resistance, they must be detailed for ductility consistent with that required for the parallel exterior shear wall or ductile moment resisting frames."

COMMENT: This proposed revision should be introduced. However, it should be noted that there appears to be an inconsistency between this proposed revision and the previous one where it was stated that the flat plate frames will not yield until story drifts greater than 0.03 h_{sx} which is larger than the maximum allowable story drift of 0.02 h_{sx}. Thus it appears there is no way that detail for ductility larger than 1 can be developed for the allowable story drift.

5.4 SEC. 4.2.2, page 56.

Alter Eq. (4-4) as follows:

"where C_T = 0.035 for steel frames or concrete frames where flat plate framing provides 35% or more of the lateral load stiffness."

COMMENT: This proposed revision should not be introduced.

REASONS: Although bare flat plate frames are very flexible, and their periods are larger than the values given by $T_a = 0.035\ C_T\ h_n^{3/4}$ when they are used in buildings the initial periods of these buildings usually decrease considerably due to the effects of the nonstructural components, even to values below those given by Eq. 4-4 using C_T = 0.035. The ATC policy in specifying values for the period to be used in estimating the seismic forces has been to specify lower values than the real ones, particularly in the case of flexible buildings, with the main objective of forcing the design of stronger and stiffer structures to avoid the large nonstructural damages that have been observed in these buildings during even moderate ground shaking. Furthermore, if it is considered that ATC allows the use of $T = 1.2\ T_a$ (see Sec. 4.2.4) and even $T = 1.4\ T_a$ (see Sec. 5.8) when T is computed according to established methods of mechanics, the acceptance of C_T = 0.035 will permit the use of T values larger than those that have been measured in the field. This is not desirable.

5.5 COMMENTARY ON SEC. 3.7.2 on page 348.

Add to last paragraph of page 348:

"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."

COMMENT: This revision should be introduced.

5.6 SEC. 3.7.12. Vertical Seismic Motions for Buildings Assigned to Categories C and D.

The vertical component of earthquake motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. For horizontal cantilever components, these effects may be satisfied by designing for a net upward force of $0.2Q_D$. For horizontal prestressed components, these effects may be satisfied by designing for a net upward force of $0.2Q_D$ if the prestressed component is not part of the seismic resisting system and bonded reinforcement not less than the minimums required by these provisions is provided where maximum moments may occur for both horizontal and vertical components of the earthquake motion. For other horizontal prestressed components, these effects may be satisfied by Formula 3-2a.

COMMENT: This proposed revision should be reviewed by Committee 2: Structural Design. In view of the little data available on the behavior of prestressed types of construction, and of the possibility that the vertical ground motions can excite inertial forces on the order of $0.5Q_D$, the writer suggests that design examples be carried out with Equation 3-2a as well as the $0.2Q_D$ net upward force suggested by Professor Hawkins.

APR 2 1980

Letter to: Ed Cohen
Gene Holland
Joe Manning
Jim Prendergast
Mark Fintel
Neil Hawkins
Vitelmo Bertero
Jim Lefter
Dick Marshall

RE: ATC 3-06 Review - Concrete Provisions

Gentlemen:

As indicated and committed in the minutes of our last meeting on 21 Feb 80 in San Francisco, enclosed are the draft provisions for precast concrete to be reviewed for inclusion in the ATC 3 document:

Add New Paragraph to Section 1.3.1

"Structures comprised of precast and/or prestressed concrete subassemblages shall be designed in accordance with the requirements of Section 11.9."

Add New Section 11.9

11.9 Structures comprised of precast and/or prestressed concrete subassemblages

The provisions of this section apply to buildings constructed with precast concrete elements not conforming to the detailing provisions given elsewhere in this section for cast-in-place concrete.

11.9.1 "Non Ductile" Construction

Structures with assemblages of precast concrete components furnishing lateral resistance against seismic forces and which do not possess energy absorption mechanisms shall be designed to resist equivalent lateral forces equal to four times the value of V_e=CsW determined from the procedure given in Section 4.2, but not to exceed V_e=1.5AvW.

Lateral load resisting walls formed by interconnecting precast elements together shall have a ratio of wall height (h) to total coupled wall length (d) - h/d of not greater than four. Walls with h/d greater than four, or where the design compressive stress exceeds $0.2f'c$ shall contain vertical boundary members in accordance with Section 11.8.4, and will not be designed under this section.

X bracing systems used to furnish lateral support for vertical load carrying frames comprised of precast and/or prestressed concrete components shall be designed to resist equivalent lateral forces equal to four times the value of V_e=CsW determined from the procedure given in Section 4.2, but not to exceed V_e=2.0AvW.

In developing the above forces, prestress strands, reinforcing, and concrete shall remain in the elastic range at the threshold of the yield point, or proportional limit state of the material.

11.9.2 "Ductile" Construction

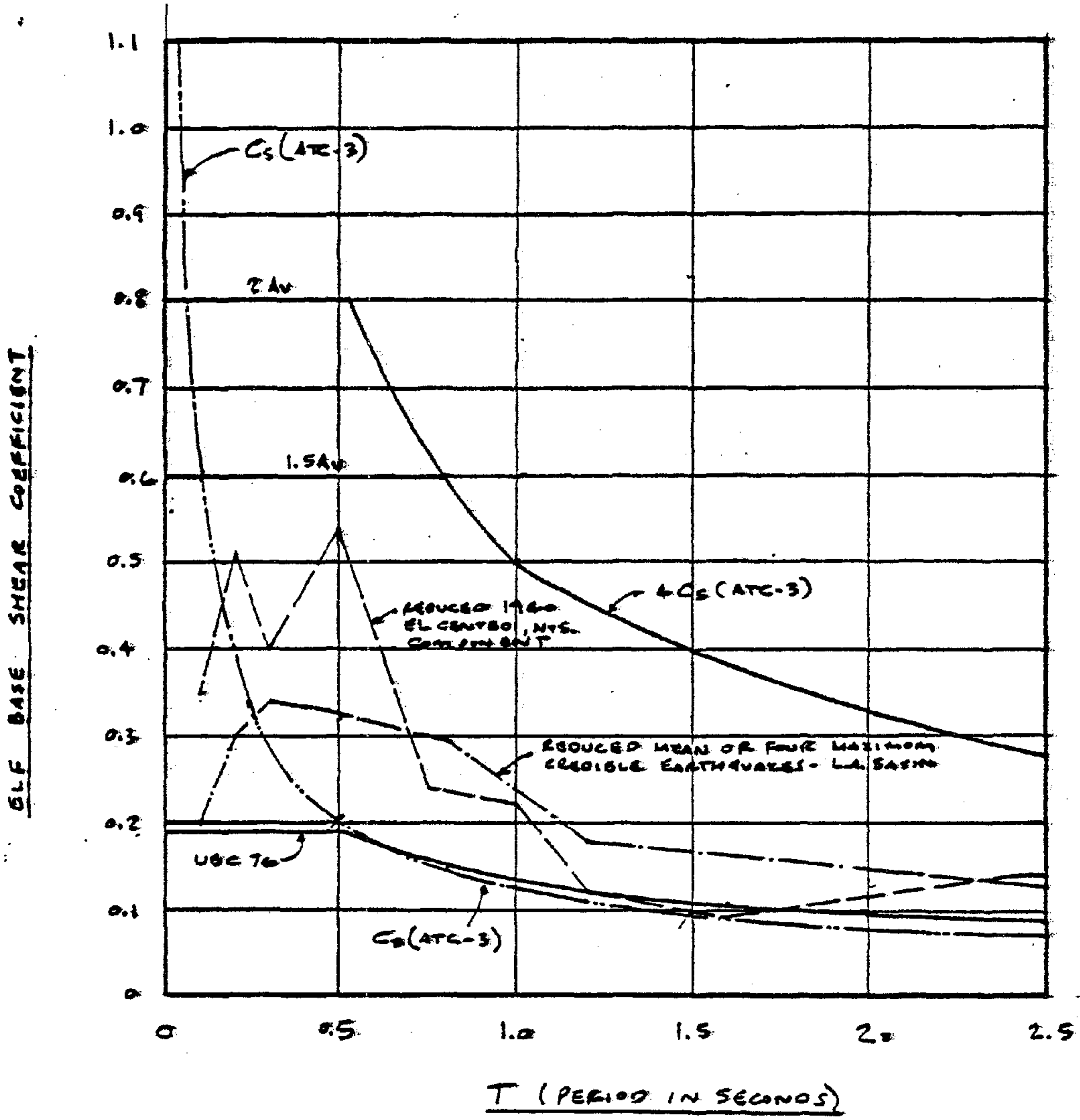
Energy absorbing lateral load resisting systems comprised of precast and/or prestressed concrete components shall be permitted provided satisfactory evidence can be shown in the form of experiments, testing, and analysis based upon established engineering principles that the resulting construction complies 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 capacities (ductility) as required from the monolithic cast-in-place ordinarily reinforced concrete construction that they replace if the R and C_d values given in Table 38 are used.

Included with these draft recommendations is a graph of equivalent lateral force values versus period, showing also the force levels generated by the 1940 El Centro earthquake, long recognized as the standard maximum credible earthquake in seismic zone 4.

Very truly yours,

David A. Sheppard
California Marketing Director

cc: Dan Jenny
California Steering Committee
PCI Seismic Committee



<u>VALUES FOR ATC-3 ELF COEFFICIENT (C_s)</u>

Aa = 0.4   Av = 0.4   SEISMICITY INDEX = 4

SEISMIC PERFORMANCE CATEGORY "C" (SEISMIC HAZARD EXPOSURE GROUP II)

R = 4.5   Cd = 4   BEARING/SHEAR WALL SYSTEM.

$\therefore C_s = 0.128\,T^{-3/2}$

<u>VALUES FOR UBC 76</u>

K = 1.33   I = 1.0   Z = 1.0   CS ≤ 0.14

8/26/80

UNIVERSITY OF CALIFORNIA, BERKELEY

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COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
DIVISION OF STRUCTURAL ENGINEERING
AND STRUCTURAL MECHANICS

BERKELEY, CALIFORNIA 94720

June 2, 1980

MEMO TO: Members of the Technical Committee 4 Concerning the Review and Refinement of ATC3-06

FROM: V. V. Bertero

REGARDING: Comments Regarding the Technical Implications of Incorporating New Appendix A into ATC3-06

I must apologize to you for the way I presented my comments on the rough draft but I have been working since Friday May 30 at 5:00 p.m. practically until 8:00 a.m. today, Monday June 2, and I do not have any more time to finish my comments or polish them. Attached I have given my conclusions and recommendations and the reasons for them.

I regret to inform you that I have presented my resignation as a representative of ATC to this committee. I have enjoyed very much working with you in the technical matters. However, lately I have not enjoyed being involved in what appears to be a struggle for power over who is to write codes regarding seismic resistant design of structures. I don't like to conduct my review under pressure. I do not agree with the way that the activities of this committee have been conducted lately.

In my memo to the secretary of the committee, dated ~~May~~ March 21, 1980, I not only requested but pleaded that the committee let me know at that time whether I should try to improve Chapter 11 or try to improve the final draft of the new Appendix A. I thought that this issue was resolved at the meeting on April14. However, I learned that the industries disallowed the vote of their representatives. I consider this unacceptable of the committee. It was not until last week that I learned that I have been requested to review the new draft of Chapter 11 that was proposed to be used to incorporate the new Appendix A into ATC and that there will be an emergency meeting of the committee on June 4. My previous committment does not allow me to attend such a meeting. I don't have the time to do a review as I would like to do. For example, I could not review the commentaries of the new Appendix A and therefore I had difficulty in going through the proposed integration of the present Chapter 11 of ATC and the draft of the new proposed Appendix A. But according to the experience gained in these last two days of working, I feel that it will be a disservice to the designers that have to do trial design runs to request that they use all these conflicting documents (ATC, ACI 318-77, the interface between Chapter 11 and Appendix A and the new Appendix A).

VVB/ed
cc: R. Sharpe
E. Leyendecker

REVIEW OF AN UPDATED CHAPTER 11 (May 28, 1980) SUBMITTED BY FINTEL

AND OF THE APPENDIX A, DATED MARCH 1980

CONCLUSIONS AND RECOMMENDATIONS

by Vitelmo V. Bertero, Representative of ATC

CONCLUSIONS: Although a complete review could not be done because of lack of time, the attached comments and observations clearly lead to the following conclusion:

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, I strongly believe it would be a mistake to introduce it in the ATC 3-06 for the TRIAL DESIGN PHASE OF ATC 3-06. 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. Considering that, even if the designers were 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, the writer believes it is not wise at this time to introduce the new chapter and appendix.

RECOMMENDATIONS: The writer agrees that Chapter 11 of ATC 3-06 needs to be updated and integrated with the new Appendix A. Therefore it is 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.

cc: Roland Sharpe
E. Leyendecker

MEMO TO: E. Cohen, Chairman of Technical Committee 4: Concrete Review and Refinement of ATC 3-06, and
E. Pfrang, Chief of Structures and Material Division, NEL

FROM: V. Bertero, Representative of ATC

RE: Technical Implications of Incorporating ACI 318-77 and New Appendix A by Reference into ATC 3-06

According to the request formulated by you through Mr. Fintel's letter of May 29, 1980, I met with Mr. Fintel and Mr. Neville, ACI Committee 318 Secretary, on Friday, May 30, 1980, at 3 p.m. in 750 Davis Hall, University of California, Berkeley, to discuss the above technical implications. As requested in the same letter, the following are my written comments. It should be noted that these comments are of a preliminary nature as I did not have time to go through the document as thoroughly as I would like since it was only delivered to me on the evening of Wednesday, May 28, 1980. For example, the provisions regarding joints of frames (Section A.6 of the new Appendix A) differs considerably from the ATC provisions on joints (Section 11.7.3). To comment properly on the implications of this change would require the technical background material (data) on which the changes have been based and the time to study it. I did not have either.

I. CHANGES NEEDED IN CHAPTER 11.

Page 1. Sec. 11.1 should read: Refs. 11.1:

[1] ANSI/ACI 318-77 "Building Code Requirements For Reinforced Concrete" but excluding Appendix A; and

[2] New proposed Appendix A - Requirements for Reinforced Concrete Building Structures Resisting Forces induced by Earthquake Motions, 19 March, 1980.

Page 2. Sec. 11.4.1 should read "Where Ordinary Moment-Resisting Frame Systems are used for the seismic-resisting system, frame components (beams, columns, and their joints) shall be proportioned to satisfy, in addition to the requirements of Chapters 1 through 17 of Ref. [1] (ANSI/ACI 318-77), the provisions A.3.2, A.3.3, A.4.3 and A.8.2 of Ref. [2]. [NEW APPENDIX A] with the following additions and exceptions:

1. A.3.2.1 Last sentence should read "At least two No. 5 or larger bars shall be provided continuously both top and bottom."

2. A new section, A.3.2.5, should be added in the new Appendix A. This section A.3.2.5 should contain the provisions required in ATC Sec. 11.6.1, paragraphs 4 and 5, i.e., "A flexural member framing . . . yield stress." "Longitudinal reinforcement . . . for the reinforcement."

3. A.4.3.2 The first sentence should read "Lap splices are permitted only within the center half of the span and shall be proportioned as tension splices. Welded . . ."

Page 3. Sec. 11.4.1 (E). Add ". . . considering the probability of full reversals of the sense of the torsional moments (torsional resistance combined with flexural under reversal moments deteriorate significantly when conventional web reinforcement is used)."

Page 3. Sec. 11.5.1. Add at the end of this section ". . . A.2.5, except that ASTM 615 Grade 60 reinforcement should not be used when welding of this reinforcement is used." (See my comments of Feb. 11, 1980.)

Page 3. Sec. 11.5.2. First paragraph, last line should be changed as follows: ". . . provisions of Ref. [2], i.e., new proposed Appendix A." Second paragraph, second line, same change as above. (The same change should be made throughout the whole proposed draft.)
In the first paragraph it is necessary to clarify that ATC refers to "braced frames" while A.5 refers to trusses. This inconsistency should be removed. I recommend that, rather than incorporating the exceptions here, a new Section 11.6.1 be added on page 4 as it was recommended be done on the May 5, 1980, ballot, i.e., a new Section 11.9 of the ATC document.

Page 3. Sec. 11.5.3. Should read "Structural walls shall have vertical boundary members which shall be proportioned to satisfy the provision A.5.3 of the New Appendix. Vertical boundary . . . can be developed. If lap splices are needed at these levels, they shall be proportioned as tension splices."

Page 4. Sec. 11.5.3. First paragraph (top of page) should be changed to read "Structural diaphragms shall be provided with boundary or edge elements at any section where tensile axial forces can be developed across the entire diaphragm section. These boundary elements shall be designed as required by A.5.3. If lap splices are needed at these sections, they shall be proportioned as tension splices."

Page 4. Sec. 11.5.4. This section should read as follows: "STRUCTURAL COMPONENTS NOT PART OF THE SEISMIC-RESISTING SYSTEM.
All structural components assumed to be not part of the seismic-resisting system shall comply with Sec. 3.3.4(C) and shall conform with the provisions of Sec. A.8 of the new Appendix A except for Sec. A.8.1. This Sec. A.8.1 does not apply to the investigation of the deformation compatibility of these components; Sec. 3.3.4(C) is the one that should be used.
The design of such components shall satisfy the minimum reinforcement requirements specified in Chapters 7, 10 and 11 of ACI 318 and Secs. A.3.2.1 and A.5.2.1. 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 provisions A.3.3 and/or A.4.4 of the new Appendix A.

II. CHANGES NEEDED IN THE NEW APPENDIX A TO MAKE IT CONSISTENT WITH THE ATC 3-06 PROVISIONS

Page 1. A.0 Notation
h - should read h"
Note that some notations are different from those of ATC. For example, h" is h_c in ATC, S is S_h in ATC, and P_j is P_n in ATC. Therefore it is

recommended the notations be reviewed thoroughly for Appendix A and ATC to assure their consistency.

Page 2. A.1 Definitions
There are discrepancies in some of the definitions used by ATC and Appendix A. For example, the definitions of cross-tie do not agree; also Structural Wall vs. Shear Wall, Structural Diaphragms vs. Diaphragm, Structural Trusses vs. Braced Frames, etc. Therefore it is recommended that the definitions in the two documents be thoroughly reviewed and the discrepancies removed.

Page 3. Definition of Anchorage Length for a Bar with a Standard Hook.
This definition does not agree with results of laboratory experiments and field inspection of damages. The effective length of anchorage cannot be counted from the critical section (where the strength of the bar which is located at the faces of the joint is to be developed). The concrete of the joint that is not confined (which has the shape of a cone) is not effective in supplying anchorage. This definition should be changed to consider the cone of unconfined concrete.

Page 3. Sec. A.2.1.1 This provision should be clarified. Limitations on the amount of energy dissipation that can be used, or would be acceptable or tolerable, should be specified. Can these provisions be used when the nonlinear response of the structure would demand "displacement ductility" of the order of 10, 20, 30, 40, 50, 60 . . . ? As is it written now, it is too vague and could lead to misuse of the provisions.

Page 4. Sec. A.2.1.3. Are the requirements for Zone 2 as defined by the UBC 1979 (I assume that it is the 1979 edition of the UBC to which this Appendix A refers) compatible with the requirements for good seismic performance of buildings assigned to Category B? This should be discussed and clarified.

Page 4. Sec. A.2.1.4. Are the requirements of the UBC 1979 for regions following Zones 3 and 4 sufficient to guarantee good seismic performance for buildings assigned to Categories C and D? This should be discussed and clarified.

Page 5. Sec. A.2.3.2. Does $\phi = 0.5$ apply only to the computation of the strength of the element under concentric axial force, or does it apply also to the combined axial force and bending moment, i.e., to the whole N-M interaction diagram for $N > A_g f'_c/10$ (as it was established in ATC)?

Page 5. Sec. A.2.5.1. A flag regarding the weldability of ASTM A615 Grade 60 should be inserted. Furthermore, it should be noted that, while ATC required that in tests the actual yield stress not exceed the specified yield stress by more than 21,000 psi (18,000 + 3,000), the new Appendix A allows 22,000 psi (18,000 + 4,000). I do not have the background material that has been used to justify this change. Note that the higher the value that is accepted, the less meaningful become the computations based on specified yielding (quality control of material is a must if we want to improve seismic-resistant design and construction).

Page 5. Sec. A.3.2.1. The last sentence should read "At least two No. 5 or larger bars shall . . ."

Page 8. Sec. A.4.2.1. This section should be modified to read as follows: "At any joint . . . the sum of the flexural strengths of the columns calculated considering the critical combination with the possible critical axial forces (whole range of possible axial forces acting in combination with the moments should be considered) shall exceed the sum of the moments at the columns obtained from the equilibrium at the joint when it is considered that the beams framing into that joint in the plane of the frame under consideration reached their flexural strength. The flexural strenghs shall be . . ."

Page 8. Sec. A.4.2.2. This section should be deleted or completely modified. Reasons: It allows the design of weak column-strong beam frames that can lead to soft story. Since the time this philosophy was proposed, I have opposed it because it leads to an unsound seismic-resistant system. It is not that the columns cannot be made ductile, but rather that the formation of a soft story leads to such large demands of energy dissipation capacity (ductility displacement demands) from the columns that these demands cannot be supplied. Therefore, it should be made clear that, except for frames of more than 2 stories, attempts should be made to prevent the development of soft stories. Any provision that will allow the formation of such soft stories should be deleted. Following this basic seismic-resistant guideline, if this section is not deleted it should be modified as follows: "A.4.2.2 - At any story level of a frame, a certain number of columns could be allowed to not satisfy Sec. A.4.2.1 provided that the remaining columns in that story of the frame complying with the requirements of Sec. A.4.2.1 are capable of elastically resisting the entire story shear at that level, accounting for the altered rigidities and torsion resulting from the omission of elastic action of the nonconforming columns. In addition, the nonconforming columns shall be provided with transverse reinforcement as specified in Sec. A.4.4 over their full height if the factored axial force in those columns exceeds $(A_g f'_c/10)$."

Page 9. Sec. A.4.3.2. At the end of the first sentence should be added ". . . span and shall be proportioned as tension splices. Welded . . ."

Page 9. Sec. A.4.4.1. In the list of notations, the following corrections should be made: Replace h with h", also in the definition of A_{s_h}. If this notation is used, the notation in ATC, pp. 40-43, should be modified also.

Page 10. Sec. A.4.4.1 Item (4). This item should be deleted as it can lead to unsound seismic-resistant practice by allowing columns without ductility since no confinement is required. Confinement of the concrete core is not only required for developing extra strength in the confined concrete required to compensate for the loss of the cover, but also to increase the deformation capacity (ductility). It is well documented through experiments and field inspection of earthquake damages that the cover of the columns at the joints will pull out and spall, reducing the effective area of concrete available to resist the internal forces to an effective cross-sectional area even smaller than that of the confined core. Application of the requirements of this Appendix does not guarantee that the column will remain elastic, because of the effects of strain hardening of beam reinforcement and the effects of higher modes of vibration. It is for these same reasons that I strongly support the recommendation in the present UBC (1979) that requires that shear strength of columns be computed based on the column core area.

The application of the provision of this section together with Sec. A.4.2.2 can lead to disaster. Therefore, I strongly recommend the deletion of these two sections or their modification.

Page10. Sec. A.4.4.4. At the end of this provision should be added "For members for which the calculated point of contraflexure is not within the middle half of their span, the special transverse reinforcement specified above should be provided over the full height of the members." (See ATC 11.7.2(B)5 (p. 106).

Page 11. Sec. A.5.2.3. What is understood by "elements of structural diaphragms" should be clarified. Are these Collector Elements and/or Boundary Elements? This should be specified. I also consider it necessary to add after the fifth line of this provision the following requirement: ". . . 0.15 f'_c, provided that no tensile forces or significant shear forces are developed simultaneously in these elements. If these elements could be subjected to significant shear forces (e.g., $v_u = 3\sqrt{f'_c}$) and to tensile forces, they shall have special transverse reinforcement as specified in Sec. A.4.4 over the total length of the element.

Page 11. Sec. A.5.3.1. The requirement should be added for the case where tensile axial forces can be developed (see 11.5.3).

Page 13. Sec. A.6.3.1. This whole provision needs clarification.
(1) It is suggested that the definition of A_j be given in the notation, Sec. A.0, or directly in this section rather than giving it in Sec. A.6.3.2. Furthermore, the definition given is not clear. What does "the design shear commentary force" mean? Should this read "shear generating force"? Should A_j be the total area, the effective area bd, or the confined core area?
(2) In lines 2 and 5 the symbol γ is missing; they should read "coefficient γ".

Personally, I question the soundness of some of these provisions (see my general comments about weaknesses in the ATC and Appendix A provisions).

Page 13. Sec. A.6.4. This section needs clarification. The value of ϕ is not given in this section. The reader has to go to the Commentary to find that ı has been defined in Sec. A.2.3.3. No indication is given of the location of the critical section for computing the development lengths ℓ_{ah} and ℓ_{as}. I personally would like to see explicitly in the equation for the estimation of the anchorage length the 1.25 f_y. This is a new section which appears able to give quite different results than those obtained according to the recommendations of Committee 352 (ACI Journal/July 1976), depending on where the critical section for anchorage is taken. I did not have the background material at hand to study this new section, but it appears to me these provisions do not properly consider the effects of deformation reversals to which the anchored bar can be subjected. The reasons follow.

(1) The commentary refers to data presented by ACI Committee 408 which does not include the effect of deformation reversals. Apparently the only attempt to account for this effect has been to specify a reduction factor of $\phi = 0.65$ rather than the $\phi = 0.80$ recommended by Committee 408. This is again a misuse of the original intent of the reduction factor ϕ.

(2) No indication is given where the critical section for anchorage should be located. The research I have conducted clearly shows that there is a core of unconfined concrete [whose depth depends on cover (shell concrete) and spacing of reinforcement in the joint core] which is ineffective in developing the reinforcement. Thus it appears to me that, if designers assume that the critical section is at the face of the joint, the application of this provision A.6.4 can lead to unconservative anchorage, particularly in the case of narrow columns.

Therefore at present I cannot support or recomment the inclusion of this provision.

Page 14. Sec. A.7.1.2. Although this section is similar to that in ATC 11.7.2(C), p. 106, I believe it is incorrect. The nominal moment strengths should be calculated for the critical axial force in the possible range of axial forces. In the selection of this critical axial force, proper N vs. M interaction diagram and the variation of the shear strength with N should be considered.

Page 14. Sec. A.7.1.3. This section cannot be used in conjunction with the ATC document. The design shear force shall be obtained from the factored loads and combinations of Sec. 3.7 of the ATC document, and not from Sec. 9.2 of ACI 318.

Page 15. Sec. A.7.3.1. The application of equation (A-5) to barbell and flanged wall cross sections is not clear because, according to the definitions of A_c and ρ_a, only the areas of concrete and steel bounded by web thickness and height of section should be considered. It appears to me that all the steel located in the edge member of the barbell shape should be considered. Similarly, all the steel located in the flange effective width of the flanged cross section should be considered.

Page 17. Sec. A.9.2.2. Equation (A-6) does not agree with equation 11-6 of ATC. Note that in (A-6) the reduction factor ϕ is missing. This appears contrary to the main philosophy of the whole ACI 318-77 document in which Required Strength $\leq \phi$ [Nominal Strength]. Furthermore, notation for the factored compressive force at the construction joint, i.e., P_j, in ATC is P_n. Therefore, a change should be made either in Sec. 2.2 Symbols of ATC or in A.0 and A.9.2 of the new Appendix A. Note the inconsistency in A.9.2 regarding the notation of this force. In equation (A-6) this force is designated as P_j but three lines below this equation (A-6) it is defined as P_n.

REVIEW OF AN UPDATED CHAPTER 11 (May 28, 1980) SUBMITTED BY FINTEL AND OF THE APPENDIX A, DATED MARCH 1980

CONCLUSIONS AND RECOMMENDATIONS

by Vitelmo V. Bertero, Representative of ATC

CONCLUSIONS: Although a complete review could not be done because of lack of time, the attached comments and observations clearly lead to the following conclusion:

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, I strongly believe it would be a mistake to introduce it in the ATC 3-06 for the TRIAL DESIGN PHASE OF ATC 3-06. 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. Considering that, even if the designers were 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, the writer believes it is not wise at this time to introduce the new chapter and appendix.

RECOMMENDATIONS: The writer agrees that Chapter 11 of ATC 3-06 needs to be updated and integrated with the new Appendix A. Therefore it is 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.

cc: Roland Sharpe
E. Leyendecker

3.4 Reference Documents

- Revised Chapter 11 - Reinforced Concrete (June 4, 1980)
- Commentary Chapter 11 - Reinforced Concrete (June 4, 1980)
- Revisions to Incorporate Revised Chapter 11 into ATC 3-06 (June 4, 1980)
- Proposed Revision of ACI 318, Appendix A - "Requirements for Reinforced Concrete Building Structures Resisting Forces Induced by Earthquake Motions" (March 19, 1980)

June 4, 1980

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 $(1 + 4/\beta_c)\sqrt{f'_c}b_od$ 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 II-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.

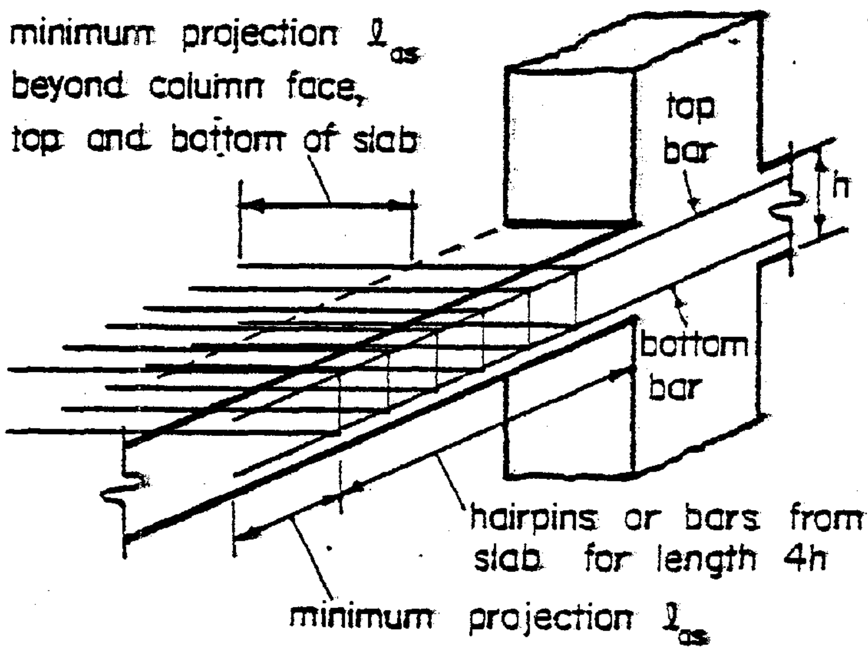

Fig. A - Reinforcement Details
Satisfying Section 11.4.1 (E)

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 - <u>SEISMIC PERFORMANCE CATEGORY C AND D</u>
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 3-8 - 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.I-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.

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*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).

$$\ell_{ah} = f_y d_b / 100 \phi \sqrt{f'_c} \qquad \text{(A-4)}$$

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.I.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.

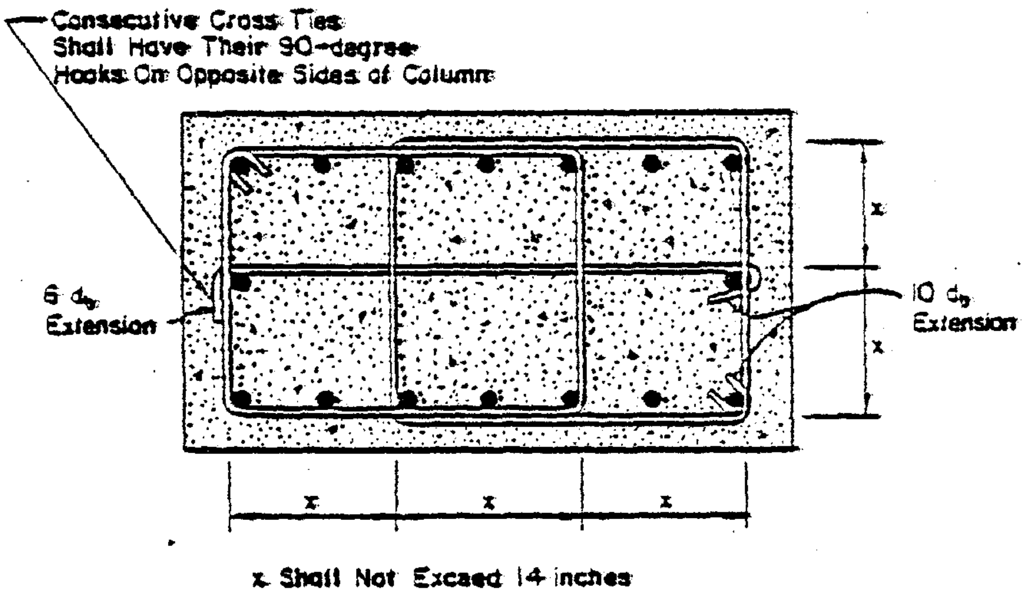


Fig. A-1

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.

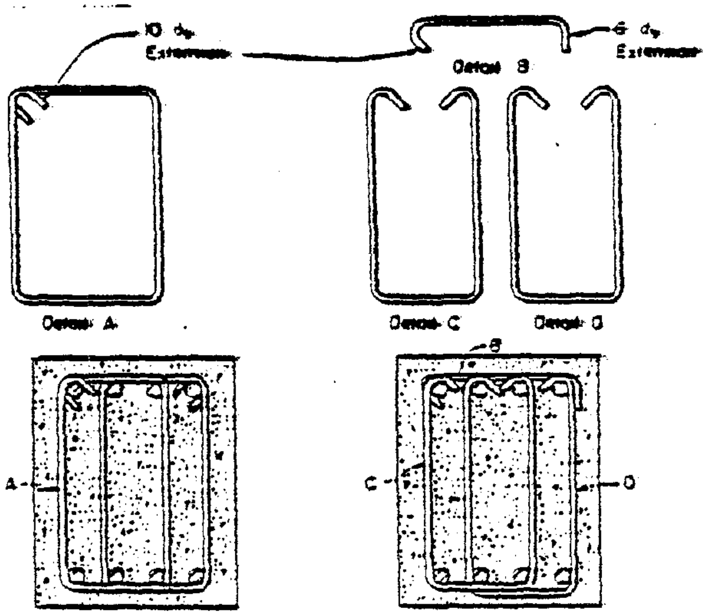


Fig. A-2

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.

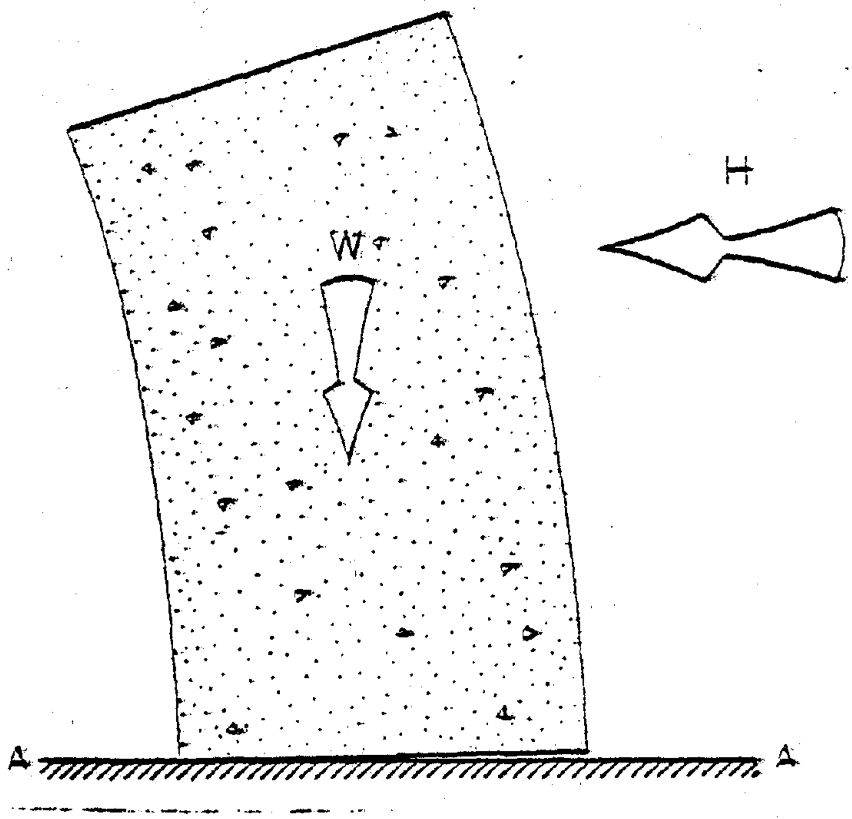


Fig. A-3

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

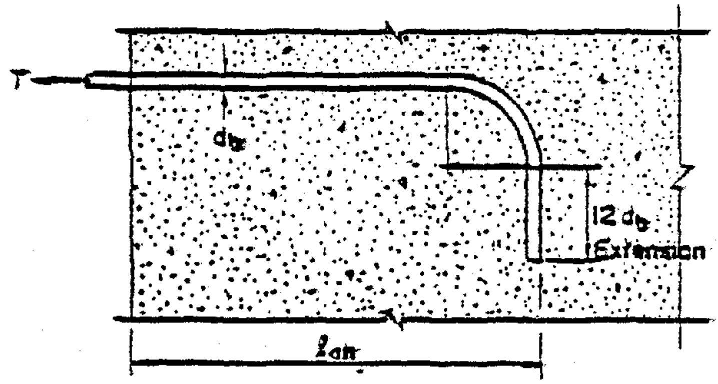
The requirements for the proportioning of joints in Appendix A are based on Reference A.12 in that behavioral phenomena within the joint are interpreted in terms of a nominal shear stress. Because tests of joints[A.19] and deep beams[A.20] indicated that shear strength was not as sensitive to joint or web reinforcement as implied by the expression developed by ACI Committee 326[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 critiical 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.

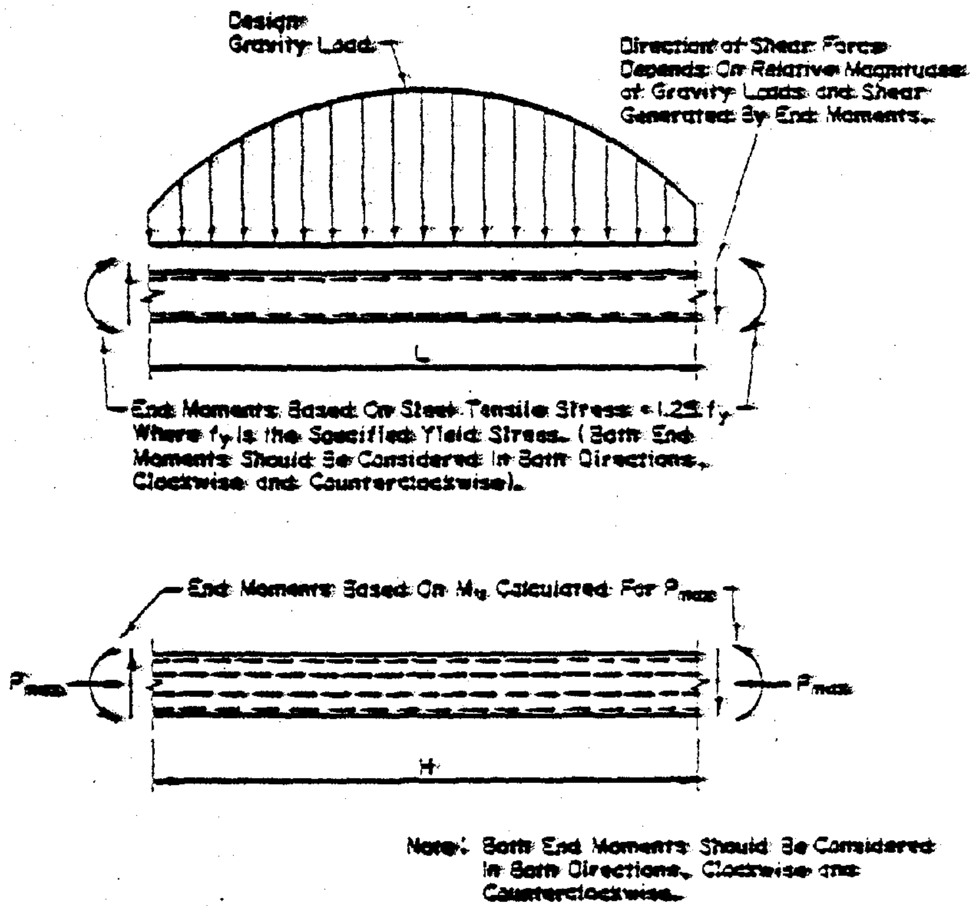


Fig. A-5

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).

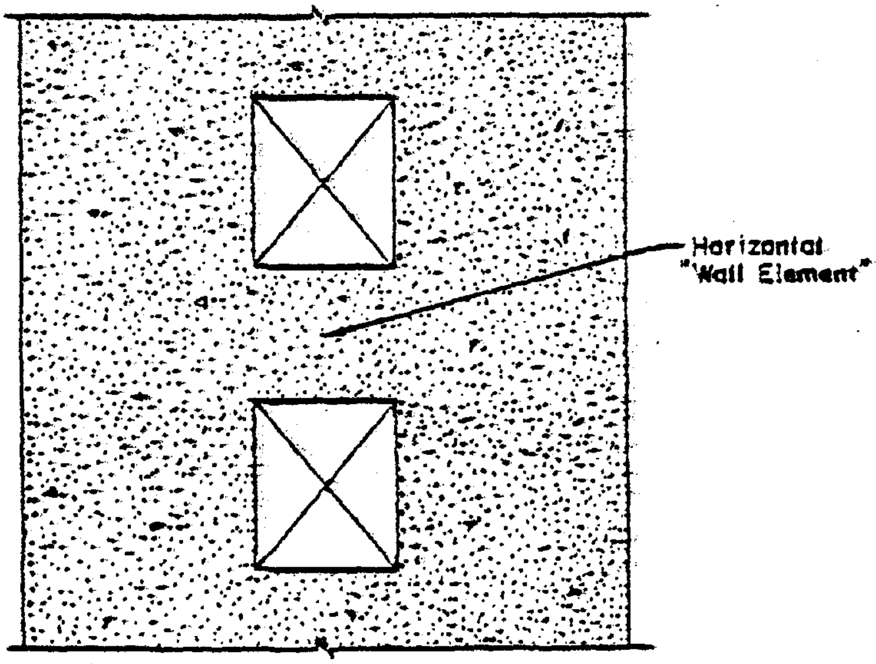


Fig. A-6

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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4.0 MINORITY REPORTS (none)

5.0 CLOSURE (none)