TENTATIVE PROVISIONS FOR THE DEVELOPMENT OF SEISMIC REGULATIONS FOR BUILDINGS

A Cooperative Effort with the Design Professions, Building Code Interests and the Research Community

Prepared by

ATC APPLIED TECHNOLOGY COUNCIL

Associated with the Structural Engineers Association of California

National Science Foundation

National Bureau of Standards

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Juanita M. Kreps, Secretary

NATIONAL BUREAU OF STANDARDS
Ernest Ambler, Director

NATIONAL SCIENCE FOUNDATION
Richard C. Atkinson, Director

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FOREWORD

Natural hazards such as earthquakes, hurricanes, tornadoes and floods cause considerable life loss and property damage in the United States each year. Improved building practices leading to buildings better able to resist extreme loads are an effective means for reducing these losses. Recognizing this, in 1972 the National Science Foundation and the National Bureau of Standards initiated a Cooperative Program in Building Practices for Disaster Mitigation. The program seeks to integrate activities and resources in Federal agencies and the work of professional organizations, private practitioners, and state and local governments in support of improved building practices.

The seismic design provisions in this report were developed as part of this Cooperative Program. The project included participation from the public and private sector. Design professionals, researchers, Federal agency representatives, staffs from model code organizations and representatives from state and local governments from throughout the United States were involved. This broad participation contributed to the success of the effort.

These provisions are tentative in nature. Their viability for the full range of applications should be established. We recommend this be done prior to their being used for regulatory purposes. Trial designs should be made for representative types of buildings from different areas of the country and detailed comparisons made with costs and hazard levels from existing design regulations. The hazard reductions achieved and the associated costs will provide useful information in considering adoption of the provisions in building regulations. In the interim, the information in this report is useful to design professionals as background to improved practice in earthquake resistant design.

Planning for such additional studies is already underway. Following completion of these additional studies, the provisions, with any modifications which may be warranted as a result of the effort, could be considered for adoption by codes and standards organizations and regulatory groups at the Federal, state, and local level.

Richard C. Atkinson, Director
National Science Foundation

Ernest Ambler, Director
National Bureau of Standards

PREFACE

This document contains tentative seismic design provisions recommended by the project participants and the Applied Technology Council (ATC) for use in the development of seismic code regulations for design and construction of buildings.

The provisions in this document prepared by the Applied Technology Council represent the result of a concerted effort by a multidisciplinary team of nationally recognized experts in earthquake engineering. ATC performed the work under a contract with the National Bureau of Standards (NBS) with funding provided by the National Science Foundation-Research Applied to National Needs Program (NSF-RANN) and NBS as part of the Cooperative Federal Program in Building Practices for Disaster Mitigation. This cooperative program integrates the extensive activities and resources of federal agencies with the work of professional organizations, private practitioners, and state and local governments in support of improved building practices. This project has involved representation from all sections of the country and has had wide review by affected building industry and regulatory groups.

BECAUSE OF THE MANY NEW CONCEPTS AND PROCEDURES INCLUDED IN THESE TENTATIVE PROVISIONS, THEY SHOULD NOT BE CONSIDERED FOR CODE ADOPTION UNTIL THEIR WORKABILITY, PRACTICABILITY, ENFORCEABILITY, AND IMPACT ON COST ARE EVALUATED BY PRODUCING AND COMPARING BUILDING DESIGNS FOR THE VARIOUS DESIGN CATEGORIES INCLUDED IN THIS DOCUMENT.

The document represents, as closely as possible, a concensus of the participants and of the reviewers. However, the publication of this document does not necessarily imply an endorsement by any individual, group, or organization, nor does it necessarily reflect the official position or policy of any individual, group, or organization.

This document is being widely distributed to the building community for review and comment. Comments will be particularly helpful in making improvements, planning follow-on studies to establish viability of the provisions, and for future use in developing seismic code regulations. All comments should be forwarded to both:

Applied Technology Council
Suite 205
480 California Avenue
Palo Alto, California 94306

Seismic Design Project
Center for Building Technology
National Bureau of Standards
Washington, DC 20234

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ABSTRACT

This document contains tentative seismic design provisions for use in the development of seismic code regulations for design and construction of buildings. The provisions represent the result of a concerted effort by a multidisciplinary team of nationally recognized experts in earthquake engineering. Design professionals, researchers, Federal agency representatives, staffs from the model code organizations and representatives from state and local governments from throughout the United States were involved. The provisions are comprehensive in nature and deal with earthquake resistant design of the structural system, architectural and non-structural elements and mechanical-electrical systems in buildings. Both new and existing buildings are included. They embody several new concepts which are significant departures from existing seismic design provisions. An extensive commentary documenting the basis for the provisions is included.

Keywords: Building; building codes; building design; disaster mitigation; earthquakes; engineering; standards.

INTRODUCTION

The basic purpose of this project is to present, in one comprehensive document, current state-of-knowledge in the fields of engineering seismology and engineering practice as it pertains to seismic design and construction of buildings. In the development of the document and its provisions, the project participants agreed upon a design philosophy and set of objectives. These are listed for guidance in understanding the goals of the project and in reading the completed document. Also listed are new concepts which in some cases are significant departures from existing seismic design provisions and areas that still require further research information. A guide to use of the provisions including a flow chart is presented immediately after this Introduction.

BACKGROUND

As population and urban densities increase in earthquake-susceptible areas, the potential of a major earthquake disaster increases. This fact has been dramatically brought to the public attention by recent earthquakes in China; Japan; Nicaragua; Guatemala; Northern Italy; Anchorage, Alaska; and San Fernando, California. Many areas of the United States, while not considered active seismically, are susceptible to major shaking; a total of 39 states may be subjected to moderate to severe earthquakes. Earthquakes, therefore, present a threat to public safety and welfare in a significant portion of the United States.

The possible level of the resultant devastation in terms of economics, social disruption, and loss of life is a matter of significant concern to government jurisdictions which have public responsibilities both before and after such disasters. In some areas of the United States where there is a recognizable potential threat of severe earthquakes occurring, there are no existing provisions controlling the seismic design and construction of buildings. In other areas present design requirements are not up to date and may be inadequate in terms of the current level of technology and experience.

The general problems pertaining to the potential threat from natural disasters were recognized in the "Disaster Preparedness Study" published in January 1972 by the U.S. Office of Emergency Preparedness as provided in Public Law 91-606. Responding to the federal government's statutory responsibility, a "Cooperative Federal Program" was initiated in 1972 by the National Science Foundation and the National Bureau of Standards following an intensive planning program made in conjunction with representatives of the Office of Emergency Preparedness and the Department of Housing and Urban Development.

The Cooperative Program established as one of its continuing objectives the following goals:

1. To synthesize current knowledge and develop improved building practices to ensure life safety in new and existing buildings.

2. To make this knowledge available in usable form to assist state and local officials in effecting land use planning and building regulations to mitigate the effects of natural disasters.

In order to facilitate these objectives a "National Workshop on Building Practices for Disaster Mitigation" to consider the effects of earthquakes, extreme winds, and other natural hazards was held at the National Bureau of Standards, Boulder, Colorado, in August 1972. The Workshop was sponsored by NSF-RANN and NBS.

Recommendations were developed at this workshop for the implementation of improved practices in the design and construction of buildings at the professional and policy-making levels. The objectives of these recommendations included the minimization of human suffering, the reduction of property losses, and the maintenance of vital functions in essential buildings subsequent to disasters. The recommendations proposed at the Workshop were the first major output of the Cooperative Federal Program. Significant among the recommendations was the development of comprehensive seismic design provisions for buildings incorporating the current state-of-knowledge.

The first task performed as a result of these recommendations was an evaluation of the response spectrum approach to the seismic design of buildings. The study was performed by ATC and funded jointly by NFS-RANN and NBS. The project provided design experience on the application of this new approach and the opportunity for a preliminary economic assessment of its effect. The results of the study are presented in a report entitled, "An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings", published by ATC in September, 1974. Following this study, ATC began work on the present document.

PHILOSOPHY

Life safety in the event of a severe earthquake is the paramount consideration in the design of buildings. With this in mind, it is intended that the provisions and principles developed by the ATC-3 project should provide an up-to-date basis for development of seismic design regulations which should enable most buildings to:

1. Resist minor earthquakes without damage.

2. Resist moderate earthquakes without significant structural damage, but with some nonstructural damage.

3. Resist major or severe earthquakes without major failure of the structural framework of the building or its component members and equipment, and to maintain life safety. It is also recognized that for certain critical facilities, particularly those essential to the public safety and well-being in case of emergency, criteria should be available to the designer which will permit design of a facility which will remain operational during and after an earthquake.

It is recognized, however, that because of the random and unpredictable nature of earthquake motions and the uncertainties concerning ultimate strength capacities and the response of buildings to earthquake motions, the seismic design requirements can not fully ensure that there will be no injury or loss of life.

OBJECTIVES

During the early stages of the ATC-3 project, much time and effort was expended to establish the project objectives. For guidance in understanding the project goals and this document, the objectives are listed below:

1. To evaluate the knowledge acquired in recent research and experience gained during on-site observations of the effects of earthquakes and to assemble it in a concise and comprehensive document for general use by building design professionals and others.

2. To write the tentative design provisions so as to permit, insofar as possible, ingenuity of solution, but with definitive criteria to evaluate the resulting design.

3. To provide seismic protection criteria which will be applicable to all probable earthquake areas of the United States.

4. To recognize that acceptable seismic risk is a matter of public policy determined by a specific government body and should be based upon:

   (a) An evaluation of available technical knowledge, including the areas of seismicity.
   (b) Reasonable means available for protection.
   (c) The magnitude of the earthquake risk compared with acceptable risks for other hazards.
   (d) The economical and social impact of a major catastrophe.

5. To provide tentative design provisions applicable to all buildings, including existing buildings, and appropriate structural as well as nonstructural components. To include requirements for structural analysis, design, and detailing which will provide adequate earthquake resistance for typical buildings and to make recommendations with respect to the design of atypical buildings.

6. To recognize that for critical facilities there should be consideration in the design of building structural and nonstructural systems of limiting damage in order to maintain the level of function determined to be necessary.

7. To provide a commentary to assist the user in understanding the intent and background of the provisions and to assess the implications of any alterations made to the provisions in the future.

NEW CONCEPTS

The provisions embody several new concepts listed below which are significant departures from existing seismic design provisions. Consequently, <u>the provisions should not be considered for code adoption</u> until a detailed evaluation is made of their workability, practicability, and potential economic impact.

1. The incorporation of more realistic seismic ground motion intensities.

2. Consideration of the effects of distant earthquakes on long-period buildings.

3. Response modification coefficients (reduction factors) which are based on consideration of the inherent toughness, amount of damping when undergoing inelastic response, and observed past performance of various types of framing systems.

4. Classification of building use-group categories into "Seismic Hazard Exposure Groups".

5. Seismic performance categories for buildings with design and analysis requirements dependent on the seismicity index and building seismic hazard exposure group.

6. Simplified structural response coefficient formulas related to the fundamental period of the seismic resisting system of the building.

7. Detailed seismic design requirements for architectural, electrical, and mechanical systems and components.

8. Materials design and analysis based upon stresses approaching yield.

9. Guidelines for systematic abatement of seismic hazards in existing buildings.

10. Guidelines for assessment of earthquake damage, strengthening or repair of damaged buildings, and potential seismic hazards in existing buildings.

AREAS REQUIRING ADDITIONAL RESEARCH INFORMATION

Although the document and its provisions embody the current state of knowledge, its development highlighted many areas that require additional research information. Most of the areas listed are broad in nature; however, the discussions in the Commentary include many of the specifics.

1. Seismic Ground Motion Intensities. This effect is incorporated in the factors A_a and A_v. Their numerical values are based on historical data and should be periodically reviewed as additional information becomes available.

2. Response Modification Coefficients, R and C_d. The numerical values of these factors are based on experience gained through the observation of the past seismic performance of various types of framing systems. These factors should be periodically reviewed as additional experience and research information are obtained.

3. The Effect of Overturning Moments. This document and recent building codes recognize that an allowance should be made for the reduction of the overturning moment for the design of the foundation and parts of the building. The amount of this reduction should be the subject of additional research.

4. Discontinuities and Irregularities. The response of a building can be significantly affected by the variation in mass and stiffness in adjacent stories and by geometric irregularities. Similarly, the response can be adversely affected by the variation in the ratio of strength provided to strength required in adjacent stories. Further research is needed in these areas, especially with regard to readily usable analysis and design procedures.

5. Orthogonal Effects. The combination of seismic forces acting in the direction of one axis combined with a portion of the seismic forces acting in the direction perpendicular to the first axis requires further investigation.

6. Torsion. The torsional response of a building can significantly affect the seismic forces induced in the structural members of a building. The manner in which these forces are included in the analysis and design of a building requires further study.

7. Calculations of the Period of a Building. The determination of the seismic design forces induced in a building using the equivalent lateral force design procedure or the modal dynamic analysis procedure is based on calculation of the fundamental building period, T. Both the approximate formulas for calculating T and the modeling techniques and material properties used in dynamic analyses require further study.

8. Diaphragms. The rigidity of a horizontal diaphragm relative to the vertical elements to which it is connected affects the distribution of the seismic forces to the vertical resisting elements and in some instances may control the seismic response of the building as a whole. Further research is required to define when a diaphragm can be classed as rigid and when the flexibility is great enough to require special consideration. This research should take into account both diaphragm openings and the stiffness of the vertical elements relative to that of the horizontal diaphragm.

9. Soil-Structure Interaction. Provisions for the effect of soil-structure interaction are included. Since this effect is the topic of many ongoing research programs, periodic review of newly developed information and its possible effects on design is essential.

10. Provisions for Structural Materials. The provisions for wood, steel, reinforced concrete, and masonry construction are based on the state of knowledge as it currently exists. Additional research data on the seismic behavior of structural components constructed from the various materials is desirable and in many cases indispensable. Ongoing research programs in many countries are producing some of this information and this must be periodically reviewed to ensure that the material provisions include the most recent data.

PROJECT ORGANIZATION

The document represents the result of a concerted effort by a multidisciplinary team of some 85 nationally recognized experts in earthquake engineering retained by ATC.

The ATC Board of Directors functioned in a general review capacity through the Executive Panel as shown in the Organization Chart of Figure A. The Executive Panel and Roland L. Sharpe, the ATC Project Director, provided overall monitoring of the project effort.

The document provisions were developed by five Task Groups with their work coordinated and guided by the Task Group Coordinating Committee (TGCC) composed of the Chairmen of the five Task Groups. The Task Groups were established on the basis of the principal subjects to be considered as follows:

1. INPUT GROUND MOTION	Risk Assessment, Ground Motion, and Site Effects
2. STRUCTURAL BEHAVIOR	Structural Design and Details, Structural Analysis, and Soil-Structure Interaction
3. NONSTRUCTURAL COMPONENTS	Architectural Systems, Mechanical/Electrical Systems and Equipment
4. LIAISON AND FORMAT	Liaison Coordination and Format Presentation
5. EXISTING BUILDINGS	Inspection and Evaluation of Damage and Potential Hazards; Repair and Strengthening

There were three groups responsible for the review of the provisions developed by the Task Groups.

A Seismic Design Review Group (SDRG) was established to monitor the development of the design provisions in terms of their technical validity, applicability to the design professions, and their appropriateness in form, substance, and use.

A Building Code Consultant Group (BCCG) was established to monitor those aspects of the document which would be applicable to code regulations and to reflect the considerations of model code groups, building officials, and public officials responsible for policymaking decisions.

The Seismology Committee of the Structural Engineers Association of California served as an independent review committee for the structural aspects of the design provisions.

In the later stages of the project the TGCC was expanded into a Project Steering Group (PSG). The PSG was given the responsibility of obtaining final Task Committee agreement and editing the final document.

During the early development of the document, three preliminary working drafts were produced for internal review. The first draft for public review was issued on January 20, 1976. Some 400 individuals and organizations received copies. The reviews received from that draft were extensive and helpful and were carefully considered by the appropriate Task Committees. A final working draft which incorporated many of the review comments was issued on January 7, 1977. A total of about 350 individuals reviewed the draft. Comments from this final review have also been carefully evaluated by the related committees and modifications made in the final document where appropriate.

Project participants are listed below. (A listing with addresses is presented in Appendix A):

PROJECT DIRECTOR

Roland L. Sharpe

EXECUTIVE PANEL

Walter D. Saunders, Chairman
Roland L. Sharpe, Vice Chairman

ATC Board

Walter D. Saunders
James L. Stratta
Stephen E. Johnston[1]
Joseph Kallaby[2]

SDRG

Henry J. Degenkolb
George W. Housner
Robert V. Whitman

BCCG

Vincent R. Bush
Christ Sanidas

[1]Until mid-1976.
[2]Executive Panel member until December 1975.

EXECUTIVE PANEL (CONT.)

Task Group Coordinating Committee

Nathan M. Newmark
Alfred Goldberg

SEAOC Seismology Committee

John Kariotis
Eric Elsesser

Ex-officio

Charles Culver, NBS
John Scalzi, NSF-RANN

TASK GROUP COORDINATING COMMITTEE LATER EXPANDED TO PROJECT STEERING GROUP (PSG)

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Nathan M. Newmark, Task Group 2
Alfred Goldberg, Task Group 3
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PSG Expansion Members

Eric Elsesser
Bruce Olsen
Clarkson Pinkham
Roland L. Sharpe
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Edwin G. Zacher

SEISMIC DESIGN REVIEW GROUP

Charles G. Culver, Chairman

J. Marx Ayres
Vitelmo V. Bertero
John A. Blume
Ray W. Clough
Henry J. Degenkolb
Neville C. Donovan
George W. Housner
Nathan M. Newmark
Richard C. Rosane, AIA
Richard O. Stone
Robert V. Whitman

BUILDING CODE CONSULTANT GROUP

Charles G. Culver, Chairman

Walter A. Brugger, Building Official
City of Los Angeles

Vincent Bush
International Conference
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Alfred Goldberg, Consulting Engineer
(Previously Building Official
City of San Francisco)

Gerald H. Jones
Building Officials Conference
of America

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Southern Building Code Congress

Norton S. Remmer, Building Commissioner
Worcester, Massachusetts

Christ Sanidas, Building Official
Shelby County, Tennessee

AD HOC REVIEW COMMITTEE SEAOC SEISMOLOGY COMMITTEE

Eric Elsesser
Norman R. Greve
John C. Kariotis
Ajit S. Virdee
Loring A. Wyllie, Jr.

TASK GROUP 1 - INPUT GROUND MOTION

Neville C. Donovan, Chairman

Risk Assessment Committee - 1A

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Soil-Structure Interaction Committee - 2C

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Jacobo Bielak
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Jose Roesset

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Elmer E. Botsai, AIA
Alfred Goldberg
G. Robert Voelz

Consultants to Task Group 3

James Dowling, AIA
Thomas D. Wosser, S.E.

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Norton S. Remmer, Chairman

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Steven J. Fenves

Design Provisions Format Committee - 4B

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Robert M. Powell

REFERENCE STANDARDS

In general National Standards are used, such as those published by ANSI, AISC, ACI, and ASTM, and the specific edition of the current standard is identified. Exceptions made in the reference standards are stated by specific section at appropriate places in the document.

ACKNOWLEDGEMENTS

Substantial review assistance in this study was provided by innumerable people including staff members of the Center for Building Technology of the National Bureau of Standards and many other federal agencies; members of professional committees of the American Concrete Institute, American Institute of Architects, American Society of Civil Engineers, and Structural Engineer Associations of Arizona, California, Colorado, Hawaii, Illinois, Iowa, Oregon, and Washington; representatives of the Building Officials Conference of America, International Conference of Building Officials and Southern Building Code Congress; numerous industry associations; and many individual engineers and architects. Numerous governmental officials and others, in response to project committee requests, furnished helpful data with respect to past experiences with earthquakes and other catastrophes. The efforts of all the above are gratefully acknowledged.

The project participants gave unstintingly of their knowledge and experience and exhibited a high degree of perseverance and cooperation in this study. Their understanding and patience contributed significantly to the success of the effort. The members of the ATC Board of Directors provided overall guidance and many helpful suggestions.

The sponsorship of the National Science Foundation, Research Applied to National Needs Program and the National Bureau of Standards made this study possible. In particular, the efforts of Dr. John Scalzi, Earthquake Engineering Program Manager, and Dr. Charles Thiel, Director, Division of Advanced Environmental Reseach and Technology, NSF and Dr. Charles Culver of the Center for Building Technology, NBS, are gratefully acknowledged.

Edwin G. Zacher was a mainstay in editing and reviewing much of the text and his assistance is gratefully acknowledged. Ronald L. Mayes, ATC Technical Director, was most helpful in coordinating technical provisions and editing. Roger E. Scholl, ATC Technical Director prior to April 1977, was most helpful with administration of the project. Lastly, Wilma Chappell typed most of this manuscript and provided helpful assistance in editing and format.

Roland L. Sharpe
Project Director

ATC-3

ORGANIZATION CHART

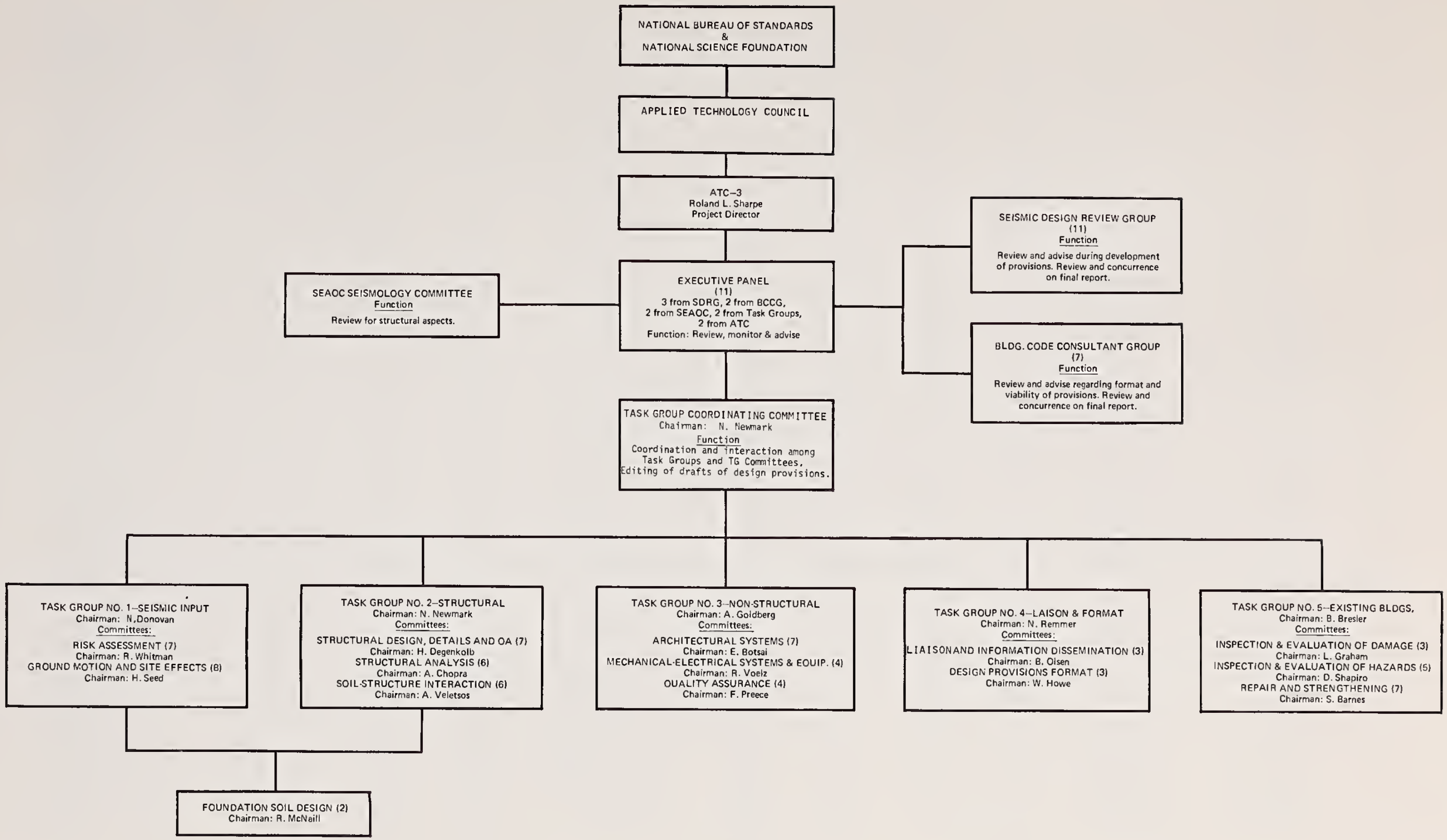


FIGURE A

GUIDE TO USE OF ATC-3 TENTATIVE PROVISIONS

The following step-by-step outline is intended to serve as a guide in the use of these Provisions for the analysis, design and detailing of buildings and their components to resist seismic forces. As a further aid, a flow chart is provided following the text.

STEP 1. Determine if proposed work is subject to provisions (Chapter 1).

a. Sec. 1.2, 1.3: New buildings, and existing buildings where alterations reducing the seismic resistance are made or where change in occupancy results in a higher assigned Seismic Performance Category (see Step 2c below), are subject to these Provisions with the following exceptions:

1) Agricultural buildings are exempt.
2) One- and two-family dwellings in areas having a Seismicity Index of 1 or 2 (see Step 2a below).

b. Special structures including bridges, industrial towers, nuclear power plants and offshore structures are not covered and require special consideration.

c. One- and two-family wood frame dwellings in areas having a Seismicity Index of 3 or 4 (see Step 2a below) need only comply with the requirements of Sec. 9.7.

d. Sec. 1.3.4: When procedures for systematic abatement of seismic hazards in existing buildings are adopted, the provisions of Chapter 13 apply.

STEP 2. Determine the required seismic performance level for the building.

a. Sec. 1.4.1: The severity of ground shaking, represented by coefficients A_a and A_v (to be used for defining design earthquake motions) and the related Seismicity Index, are determined by the procedure given, or may be specified by the jurisdiction. The Seismicity Index has values of 1 through 4; the Index of 4 is associated with the most severe ground shaking expected.

b. Sec. 1.4.2: The building is assigned to one of three Seismic Hazard Exposure Groups I, II or III, on the basis of occupancy type. Group III requires the highest level of protection.

c. Sec. 1.4.3: The building is assigned to one of four Seismic Performance Categories A, B, C or D, based on the combination of Seismicity Index and Seismic Hazard Exposure Group. Category D requires the highest level of seismic performance.

d. Sec. 1.4.4: No new building, or existing building with change in use, assigned to Seismic Performance Category D shall be sited at a potential active fault.

STEP 3. Determine general structural design requirements (Chapter 3).

a. Sec. 3.1: This section establishes the general design basis. Design forces are to be determined using the procedures of Chapters 4 or 5 (see Steps 6 and 7 below) and internal forces determined using a linearly elastic model. Members shall be sized for the forces determined, and connections generally shall develop the strength of the connected members.

The deformation of the building shall be limited. A continuous load path is to be provided, and the foundation design shall accommodate the forces and movements imparted to the building by the ground motions.

b. Sec. 3.2: Select one of three soil profile types which fits the site conditions to determine the soil coefficient, S, to be used in calculating the seismic forces.

c. Sec. 3.3: Select one of the four general framing systems representing the building. The response modification factor, R, for seismic force calculation and the deflection amplification factor, C_d, for drift determination are given as a function of the general framing system type and the vertical seismic resisting system. Requirements for Ordinary and Special Moment Frames used as seismic resisting systems are given in Chapter 10 for steel and Chapter 11 for concrete. Requirements on combinations of framing systems, and limitations for Seismic Performance Categories C and D are given in this section.

d. Sec. 3.4: The building is to be classified as regular or irregular in both plan configuration and vertical configuration. The vertical configuration determines the method of analysis to be used (see Step 4 below).

STEP 4. Select method of analysis (Sec. 3.5). The minimum level of analysis required is determined from the building Category and the regularity or irregularity of the framing system used. More rigorous methods of analysis than the minimum required may be used for any building.

a. Sec. 3.5.1: Buildings in Seismic Performance Category A need not be analyzed for seismic forces, but must satisfy the requirements given in Step 5 below.

b. Sec. 3.5.2: Buildings in Seismic Performance Category B shall as a minimum be analyzed by the Equivalent Lateral Force Procedure (Step 6 below).

c. Sec. 3.5.3: Buildings in Seismic Performance Categories C and D having regular vertical configurations shall as a minimum be analyzed by the Equivalent Lateral Force Procedure (Step 6 below). Category C and D buildings having irregular vertical configuration must be analyzed with consideration given to the dynamic properties of the building, and may conditionally use the Modal Analysis Procedure (Step 7 below).

STEP 5. Complete design requirements for Seismic Performance Category A buildings. This step summarizes all of the provisions pertinent to Seismic Performance Category A Buildings. Steps 6 through 13 are not applicable.

a. Sec. 3.6.1: The building may be constructed using any material or system permitted in Chapters 9 through 12.

b. Sec. 3.7.5: All parts of the building shall be interconnected. Minimum required capacities for ties and connections are given in this section. Foundation requirements are given in Sec. 7.3. The strength of the members and connections is determined according to Chapters 9 through 12. Sec. 12.4 gives limitations on the design of masonry components.

c. Sec. 3.7.6: Requirements for concrete and masonry wall anchorages are given in this section, and in Sec. 11.4 for reinforcement around anchor bolts in concrete.

d. Sec. 3.7.7, and Chapter 8: Architectural, mechanical and electrical components and systems shall be designed and anchored according to the requirements of Chapter 8. Note: components requiring Low Performance according to Tables 8-B and 8-C in buildings assigned to Seismic Hazard Exposure Groups I and II are exempt.

STEP 6. Determine seismic forces and displacements by the Equivalent Lateral Force Procedure (to be used for all buildings in Seismic Performance Category B and regular buildings in Seismic Performance Categories C and D).

a. Sec. 4.2: The base shear, V, shall be determined by Formula (4-1), using the seismic coefficient, C_s, given by Formula (4-2). The value of C_s to be used if the period of the building is not computed is given by Formula (4-3). Formulas for determining the period, T_a, are given. If the calculated fundamental period, T, is used, it shall not exceed 1.2 T_a.

b. Sec. 4.3: The vertical distribution of seismic forces shall be linear or parabolic, or may be interpolated between these two, depending on the period.

c. Sec. 4.4: Shear and torsion shall be distributed horizontally on the basis of relative stiffness. The magnitude of the assumed accidental torsion is given.

d. Sec. 4.5: The overturning moment may be reduced linearly between the 10th to the 20th story from the top. Limits on the reduction of the overturning moment for the design of the foundations are given.

e. Sec. 4.6: Secondary (P-delta) effects are to be considered if the stability coefficient given by Formula (4-10) exceeds 0.1. The total story drift shall be computed by using the amplication factor, C_d, for inelastic effects and the P-delta amplifications factor, if applicable.

STEP 7. Determine seismic forces and displacements by the Modal Analysis Procedure in Chapter 5 (to be used for buildings with vertical irregularities in Seismic Performance Categories C and D).

a. Sec. 5.2 to 5.4: The building shall be modeled with one degree of freedom per story, using at least the lowest three modes or all modes with a period greater than 0.4 seconds.

b. Sec. 5.5 to 5.7: The modal base shear shall be determined using the modal seismic coefficient and effective gravity load given. The modal forces and deflections shall be determined by the formulas given, and the modal shears, moments and drifts computed by linear static methods.

c. Sec. 5.8: The design values for shears, moments, and drifts shall be determined as the square root of the sum of squares of the modal quantities. If the resulting base shear is less than that determined by the Equivalent Lateral Force Procedure using T = 1.4 T_a, the design values shall be increased by the ratio of the two base shears. The base shear need not exceed that determined by Sec. 4.2.

d. Sec. 5.10: The building shall be designed to resist the design overturning moments. The calculated overturning moment may be reduced by 10 percent for the design of the foundations.

e. Sec. 5.9, 5.11: The provisions of the Equivalent Lateral Force Procedure for story shear distribution, torsion, P-delta effects and story drifts apply.

STEP 8. Determine modification due to soil-structure interaction (Chapter 6). This is an optional step. Its use will decrease the base shear and design forces, but may increase lateral deflections and secondary effects.

a. Sec. 6.2: For the Equivalent Lateral Force Procedure, the base shear is reduced using Formula (6-2), based on an effective period incorporating the rocking component and an overall damping factor including the contribution of foundation damping. The maximum reduction allowed is 0.3V. All other steps are the same as in the unmodified procedure, except that the deflections shall be modified using Formula (6-11).

b. Sec. 6.3: For the Modal Analysis Procedure, the modal base shear for the fundamental mode is reduced, and the modal deflections for the fundamental mode modified in a manner analogous to that given for the Equivalent Lateral Force Procedure.

STEP 9. Proportion and detail structural components.

a. Sec. 3.7: The load combinations to be used in determining the member and connection strengths required are given in this section. The treatment of orthogonal effects, and provisions for the effect of discontinuities in strength and lack of redundancy are given.

b. Chapters 9 through 12: The procedures to be used in determining the member and connection strengths provided are given at the beginning of each of these chapters for wood, steel, concrete and masonry.

c. Sec. 3.7 and Chapters 9 through 12: Detailing requirements necessary to insure structural integrity are given.

d. Sec. 3.6 and Chapters 9 through 12: Special design requirements for buildings in Seismic Performance Categories B, C, and D are given, including limitations on material types and framing systems that may be used.

STEP 10. Check drift limits.

a. Sec. 3.8: The drifts calculated by either the Equivalent Lateral Force Procedure or the Modal Analysis Procedure shall be limited to the allowable values given as a function of the Seismic Hazard Exposure Group, in Table 3-C.

STEP 11. Proportion and detail foundation components (Chapter 7).

a. Sec. 7.2: The procedures to be used in determining the strength of foundation members and soil capacities are given.

b. Sec. 7.4 to 7.6: Special design requirements for foundations of buildings in Seismic Performance Categories B, C and D are given.

STEP 12. Design architectural, mechanical and electrical components (Chapter 8).

a. Sec. 8.1: The general design requirements, including the definition of three levels of required performance characteristics, are given. The performance characteristic level determines the factor, P, for the required seismic force calculation.

b. Sec. 8.2: The requirements for the design of architectural components are given. The seismic forces for the design of components and their anchorages are given by Formula (8-1), which includes a seismic coefficient for the component type, C_c, and the performance factor, P. The required performance levels for the components are given in Table 8-B as a function of the Seismic Hazard Exposure Group. Additional requirements for exterior wall panel attachments, component deformation, and out-of-plane bending are given.

c. Sec. 8.3: The requirements for mechanical and electrical components are given, in a manner analogous to the architectural components. The seismic force Formula (8-2) incorporates two additional amplification factors, one based on type of attachment and one dependent on height in the building at which the component is located. The required performance levels for the components are given in Table 8-C. Additional requirements for protective devices at utility and service interfaces are given.

STEP 13. Develop and execute Quality Assurance Plan. A Quality Assurance Plan is required for buildings assigned to Seismic Hazard Exposure Groups II and III in areas with Seismicity Index of 4, and buildings in Seismic Hazard Exposure Group III in areas with Seismicity Index of 3.

a. Sec. 1.6.1: The responsibilities of the person developing the Plan and of the contractor prior to the commencement of work are given.

b. Sec. 1.6.2, 1.6.3: The work subject to special inspection and special testing as part of the Quality Assurance Plan is given.

c. Sec. 1.6.4, 1.6.5: The required inspection reporting, compliance assurance, and manufacturers' certification documentation are given.

STEP 14. Conduct systematic evaluation of seismic hazards in existing buildings (Chapter 13). These are optional provisions presented for consideration for local adoption.

a. Sec. 13.2: Inspection and evaluation procedures, including classification of buildings and qualitative and analytical evaluation procedures are given.

b. Sec. 13.3: Remedial measures for the abatement of hazards, including the time permitted for abatement, are given.

STEP 15. Develop procedures for repair and strengthening of existing buildings (Chapter 14). Guidelines are presented for consideration in local implementation.

a. Sec. 14.1 to 14.2: General concepts, including the role of building codes, economic considerations, and quality assurance requirements are described.

b. Sec. 14.3 to 14.9: Specific guidelines for repair of various structural materials and systems, foundations and non-structural elements are presented.

STEP 16. Develop and conduct emergency post-earthquake inspection and evaluation procedures (Chapter 15). These Guidelines are for consideration in local implementation.

a. Sec. 15.1 to 15.2: Procedures for developing plans and conducting damage evaluations are presented.

b. Sec. 15.3: Personnel selection, recruitment, enrollment, training and mobilization procedures are described.

c. Sec. 15.4 to 15.9: Procedures for inspection, evaluation of structural and nonstructural damage, auxiliary systems and soil conditions due to earthquake, tsunami and seiche effects are given.

d. Sec. 15.10: Procedures for reinspection and record-keeping are described. Abatement of post-earthquake hazards shall be performed in accordance with the procedures given in Chapters 13 and 14.

SI CONVERSION UNITS

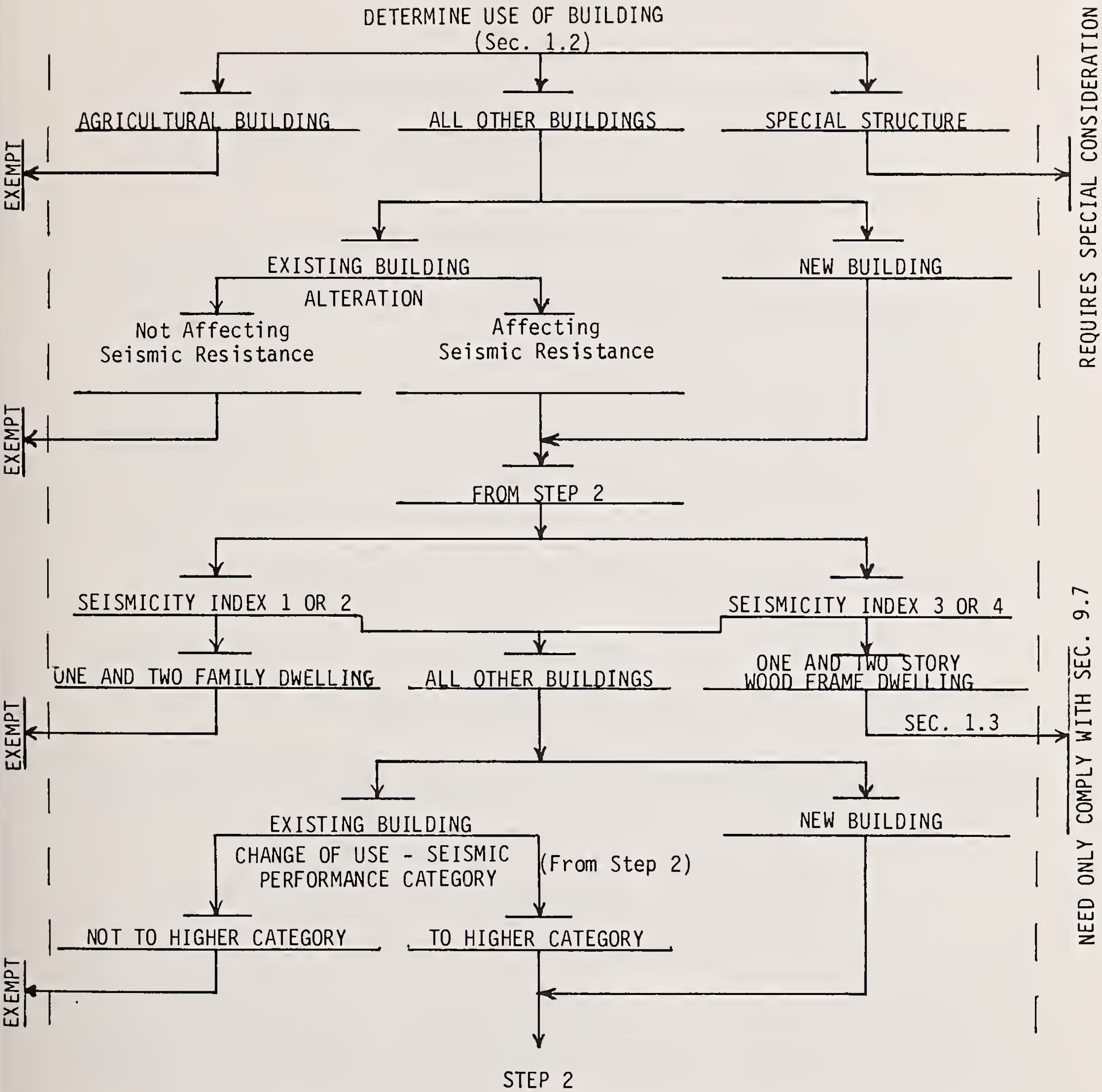
In view of the present accepted practice for building technology in this country, common U.S. units of measurements have been used throughout this publication. In recognition of the postion of the United States as a signatory to the General Conference on Weights and Measures, which gave official status to the International System of Units (SI) in 1960, the table below is presented to facilitate conversion to SI Units. Readers interested in making further use of the coherent system of SI units are referred to: NBS SP 330, 1972 Edition, The International System of Units; and ASTM E380-76, Standard for Metric Practice.

Table of Conversion Factors to SI Units

To Convert From	To	Multiply By
degree	radian	1.7453×10^{-2}
inch	meter	$2.54^* \times 10^{-2}$
in^2	m^2	$6.4516^* \times 10^{-4}$
in^3	m^3	1.6387×10^{-5}
in^4	m^4	4.1623×10^{-7}
foot	meter	$3.048^* \times 10^{-1}$
pound-force	newton	4.4482
lbf • ft	N m	1.3558
lbf/ft	N/m	1.4594×10
lbf : in	N • m	1.1298×10^{-1}
lbf/in	N/m	1.7513×10^{2}
lbf/min	N/sec	7.4137×10^{-2}
mile/hour	Km/h	1.6093
lbf/in^2	pascal	68947×10^{3}

*Exact value; others are rounded to five digits.

STEP 1. DETERMINE IF THESE PROVISIONS APPLY TO THE BUILDING



STEP 2. DETERMINE THE SEISMIC PERFORMANCE LEVEL APPLYING TO THE BUILDING

↓

DETERMINE SEISMIC PERFORMANCE CATEGORY (Sec. 1.4)

↓

LOCATE SITE ON MAPS AND READ MAP AREA NUMBERS

↓

DETERMINE A_a, A_v, AND SEISMICITY INDEX (TABLE 1-B)

↓

ASSIGN TO SEISMIC HAZARD EXPOSURE GROUP

↓

DETERMINE SEISMIC PERFORMANCE CATEGORY (TABLE 1-A)

CATEGORY A | CATEGORY B | CATEGORY C | CATEGORY D

SITE LIMITATION

NOT OVER ACTIVE FAULT

↓

STEP 3. DETERMINE GENERAL STRUCTURAL DESIGN REQUIREMENTS

↓

GENERAL DESIGN BASIS (Sec. 3.1)

↓

SELECT SOIL PROFILE WHICH FITS SITE CONDITIONS (TABLE 3-A)

↓

SELECT BUILDING TYPE (TABLE 3-B), WHICH IS DESIRED
LIMITATIONS ARE PLACED ON COMBINATIONS OF BUILDING TYPES

CATEGORIES A AND B
NO LIMITATION ON TYPE
OF FRAMING SYSTEM

CATEGORY C
LIMITATION ON HEIGHT
FOR TYPE OF SYSTEM USED

CATEGORY D
FURTHER LIMITATION ON
HEIGHT FOR TYPE OF SYSTEM

REQUIREMENTS FOR INTERACTION AND DEFORMATION EFFECTS

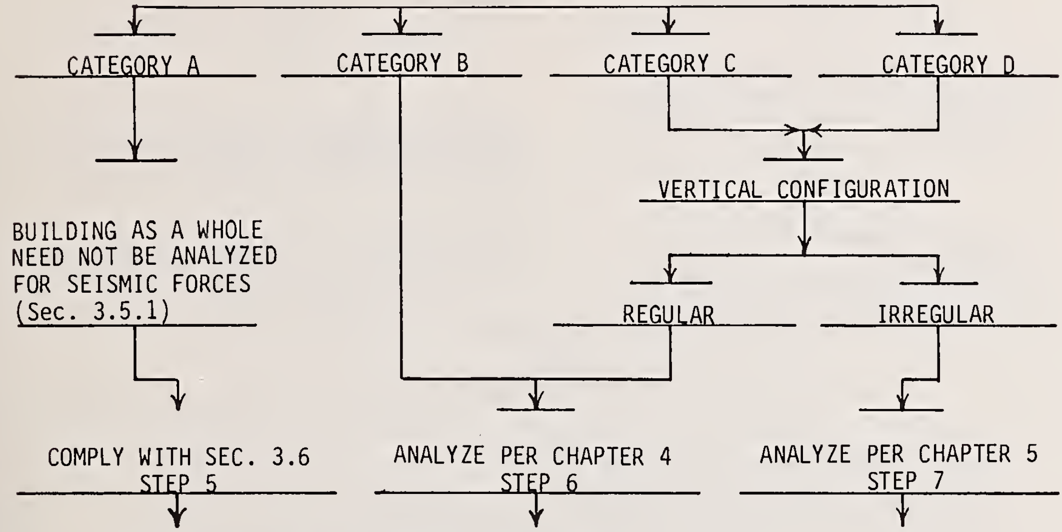
↓

CLASSIFY CONFIGURATION AS REGULAR OR IRREGULAR (Sec. 3.4)

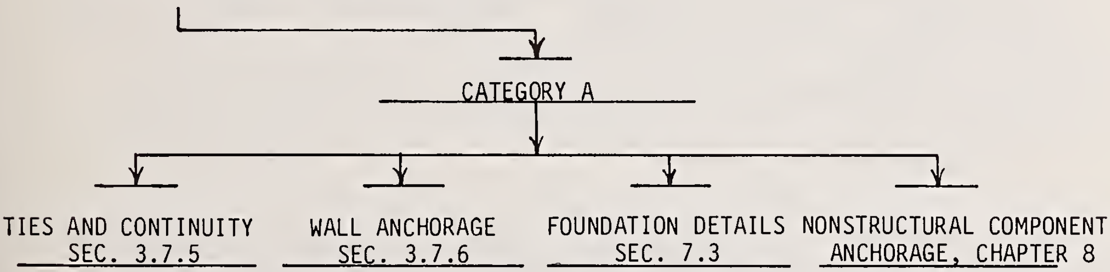
↓

STEP 4

STEP 4. SELECT METHOD OF ANALYSIS (Sec. 3.5), MINIMUM LEVEL REQUIRED



STEP 5. DESIGN AND DETAIL REQUIREMENTS



STEP 6. EQUIVALENT LATERAL FORCE PROCEDURE FOR ANALYSIS (CHAPTER 4)

BUILDING PERIOD TO BE DETERMINED

NO (Not advisable for highrise Cat. B or for Cat. C and D generally)

YES

PERIOD BY FORMULA 4-4 OR 4-5 OR FROM BUILDING PROPERTIES BUT < 1.2 T_a

C_s BY FORMULA 4-3 OR 4-3a

C_s BY FORMULA 4-2 < BY 4-3 OR 4-3a

SEISMIC BASE SHEAR V (Formula 4-1)

SOIL-STRUCTURE INTERACTION (CHAPTER 6 -- STEP 8) MAY BE USED.

VERTICAL DISTRIBUTION OF FORCES

SHEAR AND TORSION HORIZONTAL DISTRIB.

OVERTURNING MOMENTS AND REDUCTION

STORY DRIFT AND P-DELTA AFFECTS

STEP 7. MODAL PROCEDURE FOR ANALYSIS (CHAPTER 5)

MODELING PROCEDURE, NUMBER OF MODES TO BE DETERMINED, AND METHOD FOR ESTABLISHINC PERIODS

MODAL BASE SHEAR (Formula 5-1)

MODAL GRAVITY LOAD

SEISMIC COEFFICIENTS

SOIL-STRUCTURE INTERACTION MAY BE USED

MODAL FORCES, DEFLECTIONS & DRIFTS AND STORY SHEAR AND MOMENTS

DESIGN VALUES OF STORY SHEAR, MOMENT, DRIFT & DEFLECTION BY SRSS

SHEAR AND TORSION HORIZONTAL DISTRIB.

OVERTURNING AT FDN, REDUCTION

P-DELTA EFFECTS

STEP 8. SOIL-STRUCTURE INTERACTION REDUCTION (CHAPTER 6)

EQUIVALENT LATERAL FORCE PROCEDURE

REDUCTION IN BASE SHEAR BASED ON INCREASE IN PERIOD

DISPLACEMENT IS DEFLECTION + ROTATION. P-DELTA EFFECTS ARE INCREASED.

MODAL ANALYSIS PROCEDURE

REDUCTION IN FIRST MODE BASE SHEAR DUE TO INCREASE IN PERIOD

DISPLACEMENT IN FIRST MODE IS DEFLECTION + ROTATION. P-DELTA EFFECTS ARE INCREASED.

STEP 9. DETAIL REQUIREMENTS

LOAD COMBINATIONS, ORTHOGONAL EFFECTS, DISCONTINUITIES IN STRENGTH AND NONREDUNDANCY CRITERIA (Sec. 3.7)

CATEGORY B. MEET ALL CATEGORY A REQUIREMENTS

COLLECTOR ELEMENTS, DIAPHRAGMS BEARING WALLS AND DETAIL REQUIREMENTS CHAPTER 9, 10, 11, & 12

CATEGORY C. MEET ALL CATEGORY B REQUIREMENTS

VERTICAL COMPONENT OF SEISMIC MOTION & DETAIL REQUIREMENTS OF CHAPTER 9, 10, 11, & 12

CATEGORY D. MEET ALL CATEGORY C REQUIREMENTS

DETAIL REQUIREMENTS OF CHAPTER 9, 10, 11, AND 12

STEP 10. CHECK DRIFT LIMITS (Sec. 3.8)

DRIFT CALCULATED IN STEP 3 OR 4 MUST BE WITHIN LIMITS ESTABLISHED AS FUNCTION OF SEISMIC HAZARD EXPOSURE GROUP

STEP 11. FOUNDATION DETAIL REQUIREMENTS

STRUCTURAL MATERIALS AND SOIL BEARING CAPACITY CRITERIA GIVEN

CATEGORY A
NO LIMITATIONS

CATEGORY B
INVESTIGATION MAY BE
REQ'D. POLE-TYPE
STRUCTURE CRITERIA.
FDN TIES AND PILE
DETAILS

CATEGORY C
MEET CATEGORY B AND
ADDITIONAL REQMTS FOR
INVESTIGATION. FDN
TIES AND PILE DETAILS

CATEGORY D
MEET CATEGORY C.
SPECIAL PILE
LIMITATION.

STEP 12. ARCHITECTURAL, ELECTRICAL & MECHANICAL SYSTEMS AND COMPONENTS CHAPTER 8

GENERAL REQUIREMENTS, INTERRELATIONSHIP OF COMPONENTS,
CONNECTIONS AND ATTACHMENTS DESIGN REQUIREMENTS,
AND PERFORMANCE CRITERIA (Sec. 8.1 AND TABLE 8-A)

ARCHITECTURAL SYSTEMS AND COMPONENTS

FORCES (Sec. 8.2.2)

EXTER. WALL PANEL ATTACHM'T, COMPONENT
DEFORM. AND OUT OF PLANE BENDING

ELEC. & MECH. SYSTEMS AND COMPONENTS

FORCES (Sec. 8.3.2)

ATTACHMENT AND COMPONENT DESIGN
UTILITY INTERFACE

STEP 13. QUALITY ASSURANCE (Sec. 1.6)

REQUIRED FOR DESIGNATED SEISMIC SYSTEM. IS IN ADDITION TO GENERAL INSPECTION

PLAN PREPARATION
AND RESPONSIBILITY

SPECIAL INSPECTION
AND TESTING REQUIREMENTS

REPORTING AND
COMPLIANCE PROCEDURES

STEP 14. SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS (Optional) (CHAPTER 13)

FOR BUILDINGS IN CATEGORIES C AND D. PROCEDURES FOR COMPLIANCE.

EVALUATION CRITERIA
(TWO LEVELS) AND LIMITATIONS

HAZARD FOUND TO EXIST

LEVELS AND TIMES TO COMPLETE
HAZARD REDUCTION MEASURES

STEP 15. REPAIR AND STRENGTHENING OF EXISTING BUILDINGS (CHAPTER 14)

GUIDELINES FOR DIFFERENT MATERIALS AND SYSTEMS, AND LIMITATIONS ARE GIVEN

STEP 16. POST-EARTHQUAKE INSPECTION AND EVALUATION PROCEDURES (CHAPTER 15)

PROCEDURES FOR DEVELOPING PLANS FOR POST-DISASTER OPERATIONS,
EVALUATION OF DAMAGE, SELECTION OF INSPECTION PERSONNEL,
TRAINING FOR READINESS AND REPORTING AND RECORD KEEPING ARE COVERED

CHAPTER 1

ADMINISTRATION

Sec. 1.1 PURPOSE

The purpose of these provisions is to establish design and construction criteria for buildings subject to earthquake motions in order to minimize the hazard to life and improve the capability of essential facilities to function during and after an earthquake.

The design earthquake motions specified in these provisions are selected so that there is a low probability of their being exceeded during the normal lifetime expectancy of the building. Buildings and their components and elements which are designed to resist these motions and which are constructed in conformance with the requirements for framing and materials contained in the following chapters may suffer damage, but should have a low probability of collapse due to seismic-induced ground shaking.

Sec. 1.2 SCOPE

These provisions establish requirements for the design and construction of new buildings to resist the effects of earthquake motions. These provisions establish requirements for strengthening of existing buildings where alterations reducing the seismic force resistance are made or where changes in occupancy occur which would result in the assignment of the building to a higher Seismic Performance Category.

EXCEPTION:
The following buildings need not comply with these provisions:
1. Buildings classified for agricultural use and intended only for incidental human occupancy.
2. One- and two-family dwellings which are located in areas having a Seismicity Index of 1 or 2 in Table 1-B.

These provisions do not cover requirements for design and construction of special structures including, but not limited to, bridges, transmission towers, industrial towers and equipment, piers and wharves, hydraulic structures, offshore structures, and nuclear reactors. These special structures require special consideration of their response characteristics and environment which is beyond the scope of these provisions.

Sec. 1.3 APPLICATION OF PROVISIONS

New and existing buildings coming within the scope of these provisions shall be designed and constructed as required by this Section. Design documents shall be submitted to determine compliance with these provisions.

Buildings and components shall be designed for the larger of the effects due to gravity loads in combination with either other prescribed loads in the code administered by the Regulatory Agency or the seismic forces in these provisions.

1.3.1 NEW BUILDINGS

New buildings shall be designed and constructed in accordance with the applicable requirements of Chapters 2 through 12 and shall be subject to the Quality Assurance Requirements of Sec. 1.6. One- and two-story wood frame dwellings not over 35 feet in height located in areas having a Seismicity Index of 3 or 4 in Table 1-B need only conform to the requirements for Conventional Light Timber Construction as set forth in Sec. 9.7.

1.3.1 Cont.

The analysis and design of structural systems and components, including foundations, frames, walls, floors, and roofs, shall be in conformance with the applicable requirements of Chapters 3 through 7. Materials used in construction and the components made of these materials shall be designed and constructed to meet the requirements of Chapters 9 through 12. Architectural, electrical, and mechanical systems and components shall be designed in accordance with Chapter 8.

1.3.2 EXISTING BUILDING ALTERATIONS AND REPAIRS

The repair or alteration of an existing building subject to these provisions shall either (1) not reduce the lateral force resistance of the building below the requirements of these provisions or (2) shall provide for the seismic forces determined in accordance with these provisions including the modifications permitted by Sec. 13.3.

1.3.3 CHANGE OF USE

A building subject to these provisions because of a change of use shall be capable of resisting the seismic forces determined in accordance with these provisions including the modifications permitted by Sec. 13.3.

1.3.4 SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

For provisions relating to locally adopted procedures for Systematic Abatement of Seismic Hazards in existing buildings, see Chapter 13.

Sec. 1.4 SEISMIC PERFORMANCE

Seismic Performance is a measure of the degree of protection provided for the public and building occupants against the potential hazards resulting from the effects of earthquake motions on buildings. The Seismicity Index and the Seismic Hazard Exposure Group are used in assigning buildings to Seismic Performance Categories. Seismicity Index 4 is associated with the most severe ground shaking expected; Seismic Hazard Exposure Group III is associated with the uses requiring the highest level of protection; Seismic Performance Category D is assigned to provide the highest level of design performance criteria.

1.4.1 SEISMICITY INDEX AND DESIGN GROUND MOTIONS

The design ground motions are defined in terms of Effective Peak Acceleration or Effective Peak Velocity-Related Acceleration, represented by coefficients A_a and A_v, respectively. The Seismicity Index is related to the Effective Peak Velocity-Related Acceleration Coefficient. The coefficients A_a and A_v and the Seismicity Index to be used in the application of these provisions shall be determined in accordance with the following procedure:

1. Determine the appropriate map area for the building site from Figure 1-1 for A_a and Figure 1-2 for A_v.

2. Determine the value of A_a and A_v from Table 1-B for the map area found in Step 1.

3. Determine the Seismicity Index from Table 1-B for the value of A_v as determined above.

1.4.1 Cont.

ALTERNATE SECTION 1.4.1 FOR JURISDICTIONS WHICH HAVE MADE A DETERMINATION OF A_a, A_v, AND THE SEISMICITY INDEX:

"The design ground motions are defined in terms of Effective Peak Acceleration and Effective Peak Velocity-Related Acceleration, represented by coefficients A_a and A_v, respectively. The Seismicity Index is related to the effective Peak Velocity-Related Acceleration Coefficient. The coefficients A_a and A_v and the Seismicity Index to be used in the application of these provisions are established as:

A_a = _____; A_v = _____; Seismicity Index is _____."

1.4.2 SEISMIC HAZARD EXPOSURE GROUPS

All buildings shall be assigned to one of the following Seismic Hazard Exposure Groups for the purpose of these provisions:

(A) GROUP III. Seismic Hazard Exposure Group III shall be buildings having essential facilities which are necessary for post-earthquake recovery. Essential facilities, and designated systems contained therein, shall have the capacity to function during and immediately after an earthquake. Essential facilities are those which have been so designated by the Cognizant Jurisdiction. Access to essential facilities shall conform to the requirements of Sec. 1.4.2(E).

Examples of Possible Group III Facilities:

Fire suppression facilities
Police facilities
Structures housing medical facilities having surgery and emergency treatment areas
Emergency preparedness centers
Power stations or other utilities required as emergency back-up facilities

(B) GROUP II. Seismic Hazard Exposure Group II shall be buildings having a large number of occupants or buildings in which the occupants' movements are restricted or their mobility is impaired.

Examples of Possible Group II Facilities:

Public assembly for 100 or more persons
Open-air stands for 2,000 or more persons
Day care centers
Schools
Colleges
Retail stores with 5,000 sq ft floor area per floor or more than 35 feet in height
Shopping centers with covered malls, over 30,000 sq ft gross area excluding parking
Offices over 4 stories in height or more than 10,000 sq ft per floor
Hotels over 4 stories in height
Apartment houses over 4 stories in height
Emergency vehicle garages
Ambulatory health facilities
Hospital facilities other than those in Group III

1.4.2(B) Cont.

Wholesale stores over 4 stories in height
Factories over 4 stories in height
Printing plants over 4 stories in height
Hazardous occupancies consisting of flammable or toxic liquids including storage facilities for same

(C) GROUP I. Seismic Hazard Exposure Group I shall be all other buildings not classified in Group III or II.

(D) MULTIPLE USE. Buildings which have multiple uses shall be assigned the classification of the highest Seismic Hazard Exposure Group which occupies 15 percent or more of the total building area.

(E) PROTECTED ACCESS. Buildings assigned to Seismic Hazard Exposure Group III shall be accessible during and after an earthquake. Where access is through another structure that structure shall conform to the requirements for Group III. Where access is within 10 feet of side property lines, protection against potential falling hazards from the adjacent property shall be provided.

1.4.3 SEISMIC PERFORMANCE CATEGORIES

For the purposes of these provisions all buildings shall be assigned, based on the Seismicity Index established and the Seismic Hazard Exposure Group designated, to a Seismic Performance Category in accordance with Table 1-A.

Any method of analysis or type of construction required for a higher Seismic Performance Category may be used for a lower Seismic Performance Category.

1.4.4 SITE LIMITATION FOR SEISMIC DESIGN PERFORMANCE CATEGORY D

No new building or existing building which is, because of change in use, assigned to Category D shall be sited where there is the potential for an active fault to cause rupture of the ground surface at the building.

Sec. 1.5 ALTERNATE MATERIALS AND METHODS OF CONSTRUCTION

Alternate materials and methods of construction to those prescribed in these provisions may be used subject to the approval of the Regulatory Agency. Substantiating evidence demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, and seismic resistance shall be submitted.

Sec. 1.6 QUALITY ASSURANCE

This Section provides minimum requirements for Quality Assurance for Designated Seismic Systems. These requirements are in addition to the testing and inspection requirements contained in the reference standards given in Chapters 9, 10, 11, and 12.

1.6.1 QUALITY ASSURANCE PLAN

A Quality Assurance Plan shall be submitted to the Regulatory Agency for the following buildings:

Those assigned to Seismic Hazard Exposure Group II or III when located in areas having a Seismicity Index of 4, and

Those assigned to Seismic Hazard Exposure Group III when located in areas having a Seismicity Index of 2 or 3.

1.6.1 Cont.

(A) DETAILS OF QUALITY ASSURANCE PLAN. The Quality Assurance Plan shall specify the Designated Seismic Systems which are subject to quality assurance. The person responsible for the design of a Designated Seismic System shall be responsible for the portion of the Quality Assurance Plan applicable to that system. The Special Inspections and Special Tests needed to establish that the construction is in conformance with these provisions shall be included in the portion of the Quality Assurance Plan applicable to the Designated Seismic System.

(B) CONTRACTOR RESPONSIBILITY. Each contractor responsible for the construction of a Designated Seismic System, or component, listed in the Quality Assurance Plan shall submit a written statement to the Regulatory Agency prior to the commencement of work on such system or component. The statement shall clearly show the following:

1. His acknowlegement that he is aware of the special requirements contained in the Quality Assurance Plan.

2. His acknowledgement that he will exercise control to obtain conformance with the Design Documents approved by the Regulatory Agency.

3. His procedures for exercising control within his organization, the method and frequency of reporting and the distribution of the reports.

4. The person exercising such control and his position in the management of the organization.

1.6.2 SPECIAL INSPECTION

The building owner shall employ an approved Special Inspector to observe the construction of all Designated Seismic Systems in accordance with the following requirements:

(A) FOUNDATIONS. Continuous Special Inspection required during driving of piles, construction of drilled piles, and caisson work.

(B) REINFORCING STEEL. Special Inspection for reinforcing steel shall be as follows:

1. Continuous Special Inspection required during placing of steel in reinforced concrete Special Moment Frames.

2. Periodic Special Inspection required during placing of steel in reinforced concrete and reinforced masonry shear walls and Ordinary Moment Frames.

3. Continuous Special Inspection required during welding of reinforcing steel.

(C) STRUCTURAL CONCRETE. Periodic Special Inspection required during the placing of concrete in drilled piers, caissons, reinforced concrete frames, and shear walls.

(D) PRESTRESSED CONCRETE. Continuous Special Inspection required during placement of prestressing steel, during stressing and grouting operations, and during placement of concrete.

(E) STRUCTURAL MASONRY. Continuous Special Inspection required during placement of all masonry units for buildings assigned to Category D, and during all grouting operations for masonry which is part of the seismic resisting system.

1.6.2 Cont.

(F) STRUCTURAL STEEL. Continuous Special Inspection required during all shop and field welding of all multiple-pass welded connections. Periodic Special Inspection required during high-strength bolting operations for joints.

(G) STRUCTURAL WOOD. Continuous Special Inspection required during all field gluing operations. Periodic Special Inspection required for nailing, bolting, or other fastening.

(H) ARCHITECTURAL COMPONENTS. Special Inspection for Architectural Components designated in Chapter 8 as requiring S or G performance shall be as follows:

1. Periodic Special Inspection required during erection and fastening of exterior and interior architectural panels.

2. Periodic Special Inspection required during the adhesion or anchoring of veneers.

(I) MECHANICAL AND ELECTRICAL COMPONENTS. Periodic Special Inspection required during installation and anchorage of the following components when designated in Chapter 8 as requiring S or G performance.

1. Equipment using combustible energy sources.

2. Electrical motors, transformers, switchgear unit substations and motor control centers.

3. Machinery, reciprocating and rotating type.

4. Piping distribution systems 3 inches or larger.

5. Tanks, heat exchangers and pressure vessels.

1.6.3 SPECIAL TESTING

The Special Inspector shall be responsible for verifying that the special test requirements are performed by an approved testing agency for the types of work in Designated Seismic Systems listed below.

(A) REINFORCING AND PRESTRESSING STEEL. Special Testing of reinforcing and prestressing steel shall be as follows:

1. Sample at fabricator's plant and test reinforcing steel used in reinforced concrete Special Moment Frames and boundary members of reinforced concrete or reinforced masonry shear walls for limitations on weldability, elongation and actual-to-specified yield and ultimate-strength ratios.

EXCEPTION:
Certified mill test certificates may be accepted for ASTM A-706 reinforcing steel.

2. Examine certified mill test reports for each lot of prestressing steel and determine conformance with specification requirements.

(B) STRUCTURAL CONCRETE. Sample at job site and test concrete in accordance with requirements of ACI 318-71. The rate of sampling shall be at least once per day for each class placed.

(C) STRUCTURAL MASONRY. Special Testing of structural masonry shall be as follows:

1.6.3(C) Cont.

1. Sample at job site and test mortar and grout at the rate of at least once per day but not less than once for each 2,000 sq. ft. of wall area.

2. When f'_m is to be established by prism tests, at least five representative prisms shall be prepared and tested prior to start of work. During construction at least one sample prism shall be prepared per day, but not less than one sample prism per 5,000 sq ft of wall area nor less than five such sample prisms for any building during the progress of the work.

3. Sample at manufacturer's plant and test masonry units proposed for use. Sampling rate shall be at least five representative units per production lot, but not less than one unit per 5,000 sq ft of wall area. Tests shall be performed for compressive strength in accordance with ASTM Standards appropriate for the type of unit used.

(D) STRUCTURAL STEEL. Special Testing of structural steel shall be as follows:

1. Welded connections for Special Moment Frames shall be tested by nondestructive methods conforming to AWS D1.1-75. All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by other approved equivalent methods.

EXCEPTION:
The nondestructive testing rate for an individual welder may be reduced to 25 percent with the concurrence of the person responsible for structural design, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder.

2. Partial penetration groove welds when used in column splices shall be tested by ultrasonic testing or other approved equivalent method at a rate established by the person responsible for the structural design. All such welds designed to resist tension resulting from the prescribed seismic design forces shall be tested.

3. Base metal thicker than 1.5 inches when subject to through thickness weld shrinkage strains shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of criteria acceptable to the Regulatory Agency with the concurrence of the person responsible for the structural design.

(E) MECHANICAL AND ELECTRICAL EQUIPMENT. For Designated Seismic Systems or components requiring S or G performance ratings in Chapter 8, each component manufacturer shall test or analyze the component and its mounting system or anchorage as required in Chapter 8. He shall submit a certificate of compliance for review and acceptance by the person responsible for the design of the Designated Seismic System and for approval by the Regulatory Agency. The basis of certification required in Sec. 8.3.4 shall be actual test on a shaking table, by three-dimensional shock tests, or by an analytical method using dynamic characteristics and the forces from Formula 8-2, or by more rigorous analysis providing for equivalent safety. The Special Inspector shall examine the Designated Seismic System component and shall determine whether its anchorages and label conform with the certificate of compliance.

1.6.4 REPORTING AND COMPLIANCE PROCEDURES

Each Special Inspector shall furnish to the Regulatory Agency, the owner, the persons preparing the Quality Assurance Plan, and to the contractor copies of regular weekly progress reports of his observations noting thereon any uncorrected deficiencies and corrections of previously reported deficiencies. All deficiencies shall be brought to the immediate attention of the contractor for correction.

At completion of construction, each Special Inspector shall submit a final report to the Regulatory Agency certifying that all inspected work was completed substantially in accordance with approved plans and specifications. Work not in compliance shall be noted.

At completion of construction, the building contractor shall submit a final report to the Regulatory Agency certifying that all construction work incorporated into the Designated Seismic System was constructed substantially in accordance with the Design Documents and applicable workmanship requirements. Work not in compliance shall be noted.

1.6.5 APPROVED MANUFACTURERS CERTIFICATION

Each manufacturer of equipment utilized in a Designated Seismic System where the performance level required is noted in Chapter 8 as S or G shall be specifically approved by the Regulatory Agency and shall maintain an approved quality control program. Evidence of such approval shall be clearly and permanently marked on each component piece of equipment shipped to the job site.

1 Cont.

TABLE 1-A

SEISMIC PERFORMANCE CATEGORY

Seismicity Index	Seismic Hazard Exposure Group III	II	I
4	D	C	C
3	C	C	B
2	B	B	B
1	A	A	A

TABLE 1-B

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
0.15	4	0.15	3
0.10	3	0.10	2
0.05	2	0.05	2
0.05	1	0.05	1

CHAPTER 2

DEFINITIONS AND SYMBOLS

Sec. 2.1 DEFINITIONS

The following definitions provide the meaning of the terms used in these provisions.

APPENDAGE is an architectural component, such as a canopy, marquee, ornamental balcony, or statuary.

APPROVAL is the written acceptance by the Regulatory Agency of documentation which establishes the qualification of a material, system, component, procedure or person to fulfill the requirements of these Provisions for the intended use.

ARCHITECTURAL EQUIPMENT is equipment such as shelving, racks, laboratory equipment, and storage cabinets.

AREA SEPARATION PARTITION is any partition installed to provide a required fire separation between portions of buildings.

BASE is the level at which the horizontal seismic ground motions are considered to be imparted to the building.

CODE REQUIRED COMPONENT is a component required by the Building Code administered by the Regulatory Agency.

COMPONENT is a part of an architectural, electrical, mechanical, or structural system.

CONFINED REGION is that portion of a reinforced concrete component in which the concrete is confined by closely spaced special lateral reinforcement restraining the concrete in directions perpendicular to the applied stress.

CONTAINER is a large-scale independent component used as a receptacle or vessel to accommodate plants, refuse, or similar uses.

CROSS-TIE is a single No. 3 or larger bar having a 135-degree hook with a 10-diameter extension at each end.

DESIGN DOCUMENTS are the drawings, specifications, computations, reports, certifications, or other substantiation required by the Regulatory Agency to verify compliance with these provisions.

DESIGNATED SEISMIC SYSTEMS are the Seismic Resisting System and those architectural, electrical, and mechanical systems and their components which require special performance characteristics.

DIAPHRAGM is a horizontal, or nearly horizontal, system designed to transmit seismic forces to the vertical elements of the seismic resisting system.

EFFECTIVE PEAK ACCELERATION and EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION are coefficients for determining the prescribed seismic forces and are given in Sec. 1.4.

FRAME

BRACED FRAME is a vertical truss, or its equivalent, provided in a Building Frame or Dual System to resist seismic forces in the system and in which the truss members are subjected primarily to axial stress.

2.1 Cont.

BUILDING FRAME SYSTEM is a structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

DUAL SYSTEM is a structural system with an essentailly complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.

MOMENT RESISTING FRAME SYSTEM is a structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.

ORDINARY MOMENT FRAME is a Space Frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Ordinary Moment Frames shall conform to Sec. 10.4.1 or Sec. 11.6.

SPECIAL MOMENT FRAME is a Space Frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Special Moment Frames shall conform to Sec. 10.5.1 or Sec. 11.7.

SPACE FRAME is a structural system composed of interconnected members, other than bearing walls, which is capable of supporting vertical loads and may also provide resistance to seismic forces.

HIGH TEMPERATURE ENERGY SOURCE is a fluid, gas, or vapor whose temperature exceeds 220 degrees F.

HOOP is a one-piece closed tie or continuously wound tie, No. 3 or larger, which encloses the longitudinal reinforcement and has 135-degree hooks with 10-diameter extensions at each end.

JOINT is that portion of a column bounded by the highest and lowest surfaces of the other members framing into it.

JOINT, LATERALLY CONFINED is a joint which, in the direction under consideration, has the opposite faces confined by members which are monolithic with the joint and cover 75 percent of the width and depth of the joint.

LOADS

DEAD LOAD is the gravity load due to the weight of all permanent structural and nonstructural components of a building, such as walls, floors, roofs, and fixed service equipment.

GRAVITY LOAD, W, is as defined in Sec. 4.2.

LIVE LOAD is the load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load. The live load may be reduced for tributary area, as permitted by the Building Code administered by the Regulatory Agency.

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

P-DELTA EFFECT is the secondary effect on shears and moments of frame members due to the action of the vertical loads induced by displacement of the building frame resulting from seismic forces.

QUALITY ASSURANCE PLAN is a detailed written procedure which establishes the systems and components subject to Special Inspection and testing. The type and frequency of testing and the extent and duration of Special Inspection are given in the Quality Assurance Plan.

RESILIENT MOUNTING SYSTEM is a system incorporating helical springs, air cushions, rubber-in-shear mounts or fiber-in-shear mounts or other comparable approved systems.

RESILIENT MOUNTING SYSTEM, STABLE is a system in which the force displacement ratios are equal in the horizontal and vertical directions.

RESTRAINING DEVICE is a device used to limit the vertical or horizontal movement of the mounting system due to earthquake motions.

RESTRAINING DEVICE, ELASTIC, is a fixed restraining device which incorporates an elastic element to reduce the seismic forces transmitted to the structure due to impact from the resilient mounting system.

RESTRAINING DEVICE, FIXED is a non-yielding or rigid type of restraining device.

RESTRAINING DEVICE, SEISMIC ACTIVATED, is an interactive restraining device which is activated by earthquake motion.

ROOFING UNIT is a unit of roofing material weighing more than one pound.

SEISMIC FORCES are the assumed forces prescribed herein, related to the response of the building to earthquake motions, to be used in the design of the building and its components.

SEISMIC HAZARD EXPOSURE GROUP is a classification assigned to a building based on its use as defined in Sec. 1.4.

SEISMIC PERFORMANCE CATEGORY is a classification assigned to a building as defined in Sec. 1.4.

SEISMIC RESISTING SYSTEM is that part of the structural system which has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

SEISMICITY INDEX is an identification number related to the expected severity of earthquake ground motion as defined in Sec. 1.4.

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

SPECIAL LATERAL REINFORCEMENT is composed of spirals, closed stirrups or hoops and supplementary cross ties provided to restrain the concrete and qualify the portion of the component, where used, as a Confined Region.

SPECIAL INSPECTION is the observation of the work by a Special Inspector to determine compliance with the approved Design Documents and these provisions.

2.1 Cont.

SPECIAL INSPECTION, CONTINUOUS, is the full-time observation of the work by an approved Special Inspector who is present in the area where the work is being performed.

SPECIAL INSPECTION, PERIODIC, is the part-time or intermittent observation of the work by an approved Special Inspector who is present in the area where work has been or is being performed.

SPECIAL INSPECTOR is a person approved by the Regulatory Agency as being qualified to perform Special Inspection required by the approved Quality Assurance Plan.

TESTING AGENCY is a company or corporation that provides testing and/or inspection services. The person in responsible charge of the Special Inspector(s) and the testing services shall be an engineer licensed by the State to practice as such in the applicable discipline.

UTILITY OR SERVICE INTERFACE is the connection of the building's mechanical and electrical distribution systems to the utility or service company distribution system.

VENEERS are facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

WALL is a component, usually placed vertically, used to enclose or divide space.

BEARING WALL is a wall providing support for vertical loads and may be exterior or interior.

BEARING WALL SYSTEM is a structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear walls or braced frames.

NONBEARING WALL is a wall which does not provide support for vertical loads other than its own weight or as permitted by the Building Code administered by the Cognizant Jurisdiction. It may be an exterior or interior wall.

SHEAR WALL is a wall, bearing or nonbearing, designed to resist seismic forces acting in the plane of the wall.

Sec. 2.2 SYMBOLS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The following symbols and their definitions apply to these provisions.

A_a = The seismic coefficient representing the Effective Peak Acceleration as determined in Sec. 1.4.1.

A_{ch} = Cross-sectional area of a component measured to the outside of the Special Lateral Reinforcement.

A_{sh} = Total cross-sectional area of hoop reinforcement, including supplementary cross-ties, having a spacing of s_h and crossing a section with a core dimension of h_c (square inches).

A_o = The area of the load-carrying foundation .

A_v = The seismic coefficient representing the Effective Peak Velocity-Related Acceleration as determined in Sec. 1.4.1.

2.2 Cont.

a_c	= The amplification factor related to the response of a system or component as affected by the type of seismic attachment, determined in Sec. 8.3.2.
a_d	= The incremental factor related to P-delta effects in Sec. 4.6.2.
a_x	= The amplification factor at level x related to the variation of the response in the height of the building, Sec. 8.3.2.
C_c	= The seismic coefficient for components of buildings as specified in Tables 8-B and 8-C (dimensionless).
C_d	= The deflection amplification factor as given in Table 3-B.
C_s	= The seismic design coefficient determined in Sec. 4.2 (dimensionless).
$\tilde{C}_s$	= The seismic design coefficient determined in Sec. 6.2.1 and 6.3.1 (dimensionless).
C_{sm}	= The modal seismic design coefficient determined in Sec. 5.5 (dimensionless).
C_T	= The framing coefficient in Sec. 4.2.2.
C_{vx}	= The vertical distribution factor as determined in Sec. 4.2.
D_s	= The total depth of the stratum in Formula 6-10.
F_i, F_n, F_x	= The portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Sec. 4.3.
F_p	= The seismic force acting on a component of a building as determined in Sec. 3.7, 8.2, or 8.3.
F_{xm}	= The portion of the seismic base shear, V_m, induced at level x as determined in Sec. 5.6.
f_{yh}	= The specified yield stress of the Special Lateral Reinforcement, psi.
G	= $\gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels.
G_o	= $\gamma v_{so}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels.
g	= The acceleration due to gravity.
$\bar{h}$	= The effective height of the building as determined in Sec. 6.2 or 6.3.
h_c	= The core dimension of a component measured to the outside of the Special Lateral Reinforcement.
h_i, h_n, h_x	= The height above the base to level, i, n, or x, respectively.
h_{sx}	= The story height below level x, = $(h_x - h_{x-1})$.

I_o = The static moment of inertia of the load-carrying foundation, Sec. 6.2.1.

i = The building level referred to by the subscript i. i = 1 designates the first level above the base.

K = The stiffness of the equipment support attachment, Sec. 8.3.2.

K_y = The lateral stiffness of the foundation as defined in Sec. 6.2.

K_θ = The rocking stiffness of the foundation as defined in Sec. 6.2.

k = The distribution coefficient given in Sec. 4.3.

$\overline{k}$ = The stiffness of the building as determined in Sec. 6.2.

L = The overall length of the building (in feet) at the base in the direction being analyzed.

L_o = The overall length of the side of the foundation in the direction being analyzed, Sec. 6.2.1.

M_f = The foundation overturning design moment as defined in Sec. 4.5.

M_o,M_{o1} = The overturning moment at the foundation-soil interface as determined in Sec. 6.2.3 and 6.3.2.

M_t = The torsional moment resulting from the location of the building masses, Sec. 4.4.

M_{ta} = The accidental torsional moment as determined in Sec. 4.4.

M_x = The building overturning design moment at level x as defined in Sec. 4.5 or Sec. 5.8.

m = A subscript denoting the mode of vibration under consideration; i.e., m = 1 for the fundamental mode.

n = Designates the level which is uppermost in the main portion of the building.

P = The performance criteria factor as given in Table 8-A (dimensionless).

P_n = The algebraic sum of the seismic forces and the minimum gravity loads on the joint surface acting simultaneously with the shear, Sec. 11.8.7.

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

Q_D = The effect of dead load.

Q_E = The effect of seismic (earthquake-induced) forces.

Q_L = The effect of live load.

Q_S = The effect of snow load.

R = The seismic response modification coefficient as given in Table 3-B.

r = A characteristic length of the foundation, defined in Sec. 6.2.1.

2.2 Cont.

r_a	= The characteristic foundation length defined by Formula 6-7.
r_m	= The characteristic foundation length as defined by Formula 6-8.
S	= The seismic coefficient for the soil profile characteristics of the site as given in Table 3-A.
S_1, S_2, S_3	= The Soil Profile Types as defined in Sec. 3.2.
s_h	= Spacing of Special Lateral Reinforcement.
T	= The fundamental period of the building as determined in Sec. 4.2.2.
$\tilde{T}, \tilde{T}_1$	= The effective fundamental period of the building as determined in Sec. 6.2.1 and 6.3.1.
T_a	= The approximate fundamental period of the building as determined in Sec. 4.2.2.
T_c	= The fundamental period of the component and its attachment.
T_m	= The modal period of vibration of the m^{th} mode of the building as determined in Chapter 5.
V_t	= The design value of the seismic base shear as determined in Sec. 5.8.
V_x	= The seismic shear force at any level as determined in Sec. 4.4 or Sec. 5.8.
$\tilde{V}_1$	= The portion of the seismic base shear, $\tilde{V}$, contributed by the fundamental mode, Sec. 6.3.
ΔV	= The reduction in V as determined in Sec. 6.2.
ΔV_1	= The reduction in V_1 as determined in Sec. 6.3.
v_s	= The average shear wave velocity for the soils beneath the foundation at large strain levels, Sec. 6.2.
v_{so}	= The average shear wave velocity for the soils beneath the foundation at small strain levels, Sec. 6.2.
W	= The total gravity load of the building as defined in Sec. 4.2
$\overline{W}$	= The effective gravity load of the building as defined in Sec. 6.2 and 6.3.
$\overline{W}_m$	= The effective modal gravity load determined in accordance with Formula 5-2.

W_c = The gravity load of a component of the building.

w_i, w_n, w_x = The portion of W which is located at or assigned to level i, n, or x, respectively.

x = The level under consideration. x = 1 designates the first level above the base.

α = The relative weight density of the structure and the soil as determined in Sec. 6.2.1.

$\tilde{\beta}$ = The fraction of critical damping for the coupled structure-foundation system, determined in Sec. 6.2.1.

β_o = The foundation damping factor as specified in Sec. 6.2.1.

γ = The average unit weight of soil.

Δ = The design story drift as determined in Sec. 4.6.1 or 4.6.2.

Δ_a = The allowable story drift as specified in Sec. 3.8.

Δ_m = The design modal story drift determined in Sec. 5.6.

δ_x = The deflection at level x, Formula 4-9.

δ_{xe} = The deflection at level x, determined by an elastic analysis, Sec. 4.6.1.

δ_{xem} = The modal deflection at level x determined by an elastic analysis, Sec. 5.6.

δ_{xm}, $\tilde{\delta}_{xm}$ = The modal deflection at level x as determined by Formula 5-5 and 6-15.

$\tilde{\delta}_x$, $\tilde{\delta}_{x1}$ = The deflection at level x, Formula 6-11 and 6-14.

θ = The stability coefficient for P-delta effects as determined in Sec. 4.6.2.

κ = The overturning moment reduction factor, Formula 4-6.

ϕ = The capacity reduction factor.

ϕ_{im} = The displacement amplitude at the i^{th} level of the building for the fixed base condition when vibrating in its m^{th} mode, Sec. 5.5.

CHAPTER 3

STRUCTURAL DESIGN REQUIREMENTS

Sec. 3.1 DESIGN BASIS

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

A continuous load path, or paths, with adequate strength and stiffness, shall be provided which will transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to accommodate the forces developed or the movements imparted to the building by the design ground motions. In the determination of the foundation design criteria, special recognition shall be given to the dynamic nature of the forces, the expected ground motions, and the design basis for strength and ductility of the structure.

Sec. 3.2 SITE EFFECTS

Soil profile types and site coefficients, S, are given in this Section.

3.2.1 SOIL PROFILE TYPES

The effects of site conditions on building response shall be established based on soil profile types defined as follows:

SOIL PROFILE TYPE S_1 is a profile with:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity greater than 2,500 feet per second, or

2. Stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE S_2 is a profile with deep cohesionless or stiff clay conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE S_3 is a profile with soft- to medium-stiff clays and sands, characterized by 30 feet or more of soft- to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 shall be used.

3.2.2 SITE COEFFICIENT

S is a coefficient for the effects of the site conditions on building response and is given in Table 3-A.

3.2 Cont.

3.2.3 SOIL-STRUCTURE INTERACTION

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

Sec. 3.3 FRAMING SYSTEMS

Four types of general framing systems are recognized for purposes of these provisions as shown in Table 3-B. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in Sec. 3.6 and in Chapters 9, 10, 11, and 12 for buildings assigned to the various seismic performance categories.

3.3.1 CLASSIFICATION OF FRAMING SYSTEMS

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

3.3.2 COMBINATIONS OF FRAMING SYSTEMS

Where combinations of framing systems are incorporated into the same building the following requirements shall be fulfilled:

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

EXCEPTION:
This requirement need not apply to supported systems with a weight equal to or less than 10 percent of the weight of the building.

(B) DETAILING REQUIREMENTS. For components common to systems having different R values, the detailing requirements which are required by the higher R value shall be used.

3.3.3 SEISMIC PERFORMANCE CATEGORIES A AND B

Any type of building framing system permitted in these provisions may be used for buildings assigned to Categories A and B.

3.3.4 SEISMIC PERFORMANCE CATEGORY C

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

(A) SEISMIC RESISTING SYSTEMS. Seismic resisting systems in buildings over 160 feet in height shall be one of the following:

1. Moment resisting frame system with Special Moment Frames.

2. A Dual System

3. A system with structural steel or cast-in-place concrete braced frames or shear walls in which there are braced frames or walls so arranged that braced frames or walls in any plane resist no more than 33 percent of the seismic design force including torsional effects; this system is limited to buildings not over 240 feet in height.

3.3.4 Cont.

(B) INTERACTION EFFECTS. Moment resisting space frames which are enclosed by, or adjoined by, more rigid elements not considered to be part of the seismic resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force resisting capability of the space frame. The design shall consider and provide for the effect of these rigid elements on the structural system at building deformations corresponding to the design story drift Δ as determined per Sec. 4.6.

(C) DEFORMATIONAL COMPATIBILITY. All structural elements not considered in the design to be part of the seismic resisting system shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the design story drift Δ as determined in accordance with Sec. 4.6.

(D) SPECIAL MOMENT FRAMES. A Special Moment Frame which is used, but not required by these provisions, may be discontinued and supported by a more rigid system with a lower R value subject to the requirements in Sec. 3.7.3.

A Special Moment Frame which is required by these provisions shall be continued down to the foundation.

3.3.5 SEISMIC PERFORMANCE CATEGORY D

The framing systems of buildings assigned to Category D shall conform to the requirements for Category C and to the additional requirements and limitations of this Section.

The height limitations of Sec. 3.3.4 shall be reduced from 160 feet to 100 feet and for braced frame or shear wall systems the maximum height shall be reduced from 240 feet to 160 feet.

Sec. 3.4 BUILDING CONFIGURATION

For purposes of seismic design, buildings shall be classifed as regular or irregular as specified in this Section. Both plan and vertical configurations of a building shall be considered when determining whether a building is to be classified as regular or irregular.

Buildings which have an approximately symmetrical geometric configuration and which have the building mass and seismic resisting system nearly coincident shall be classified as regular.

3.4.1 PLAN CONFIGURATION

For purposes of determining diaphragm component forces and distribution of seismic forces to vertical components of the seismic resisting system, a building shall be classified as irregular when any of the following occurs:

1. The building does not have an approximately symmetrical geometric configuration or has re-entrant corners with significant dimensions.

2. There is the potential for large torsional moments because there is significant eccentricity between the seismic resisting system and the mass tributary to any level.

3. The diaphragm at any single level has significant changes in strength or stiffness.

3.4 Cont.

3.4.2 VERTICAL CONFIGURATION

For purposes of selecting an analysis procedure for determining seismic forces and the distribution of these forces, a building shall be classified as irregular when any of the following occurs:

1. The building does not have an approximately symmetrical geometric configuration about the vertical axes or has horizontal offsets with significant dimensions.

2. The mass-stiffness ratios between adjacent stories varies significantly.

Sec. 3.5 ANALYSIS PROCEDURES

This Section prescribes the minimum analysis procedure to be followed. A more rigorous generally accepted procedure may be used in lieu of the minimum applicable procedure. In no case shall the alternate procedure use fundamental building periods greater than permitted in Chapter 4 or Chapter 5.

3.5.1 SEISMIC PERFORMANCE CATEGORY A

Regular or irregular buildings assigned to Category A need not be analyzed for seismic forces for the building as a whole. The provisions of Sec. 3.6. shall apply to the components indicated therein.

3.5.2 SEISMIC PERFORMANCE CATEGORY B

Regular or irregular buildings assigned to Category B shall be as a minimum analyzed in accordance with the procedures given in Chapter 4.

3.5.3 SEISMIC PERFORMANCE CATEGORIES C AND D

Buildings classified as regular and assigned to Category C or D shall, as a minimum, be analyzed in accordance with the procedures given in Chapter 4. Buildings classified as irregular and assigned to Category C or D shall be analyzed with special consideration of the dynamic characteristics of the building. For buildings having only vertical irregularities, this requirement may be satisfied by the use of the procedures given in Chapter 5.

Sec. 3.6 DESIGN AND DETAILING REQUIREMENTS

The design and detailing of components of the seismic resisting system and of other structural and nonstructural components shall be as specified in this Section.

3.6.1 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be constructed using any material or system permitted in Chapters 7, 9, 10, 11, or 12. These buildings need only comply with the minimum seismic force requirements of Sec. 3.7.5 and 3.7.6, and to the requirements of Sec. 3.7.7 and 7.3.

3.6.2 SEISMIC PERFORMANCE CATEGORY B

Buildings assigned to Category B shall conform to the requirements for Category A and to the following requirements and limitations.

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

3.6.2 Cont.

(B) MATERIALS. The materials, and the systems composed of those materials, shall conform to the requirements and limitations in Chapters 9, 10, 11, and 12 for Category B.

(C) OPENINGS. Where openings occur in shear walls or diaphragms or other plate-like elements, chords shall be provided at the edges of the openings to resist the local stresses created by the presence of the opening. These chords shall extend into the body of the wall or diaphragm a distance sufficient to develop and distribute the stress of the chord member.

3.6.3 SEISMIC PERFORMANCE CATEGORY C

Buildings assigned to Category C shall conform to the requirements for Category B and to the following requirements and limitations.

(A) COMPONENTS. Components of the seismic resisting system and other structural components shall also conform to the requirements of Sec. 3.7.12. and 7.5.

(B) MATERIALS. The materials, and the systems composed of these materials, shall conform to the requirements and limitations in Chapters 9, 10, 11, and 12 for Category C.

3.6.4 SEISMIC PERFORMANCE CATEGORY D

Buildings assigned to Category D shall conform to the following requirements and limitations.

The materials, and the systems composed of those materials, shall conform to the requirements and limitations of Chapters 7, 9, 10, 11, and 12 for Category D.

Sec. 3.7 STRUCTURAL COMPONENT LOAD EFFECTS

All building components shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, live, and snow loads. The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. The second order effects shall be included where applicable.

3.7.1 COMBINATION OF LOAD EFFECTS

The effects on the building and its components due to gravity loads and seismic forces shall be combined in accordance with Formula 3-1 or, as applicable, 3-2 or 3-2a.

$$\text{Combination of load effects} = 1.2Q_D + 1.0Q_L + 1.0Q_S \pm 1.0Q_E \qquad (3\text{-}1)$$

$$\text{Combination of load effects} = 0.8Q_D \pm 1.0Q_E \qquad (3\text{-}2)$$

for partial penetration welded steel column splices or for unreinforced masonry and other brittle materials, systems, and connections:

$$\text{Combination of load effects} = 0.5Q_D \pm 1.0Q_E \qquad (3\text{-}2a)$$

3.7.2 ORTHOGONAL EFFECTS

The critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used.

3.7.3 DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEM

The design of a building shall consider the potential adverse effects when the ratio of the strength provided in any story to the strength required is significantly less than that ratio for the story immediately above and the strengths shall be adjusted to compensate for this effect.

3.7.4 NONREDUNDANT SYSTEMS

The design of a building shall consider the potentially adverse effect that the failure of a single member, connection, or component would have on the stability of the building and appropriate design modifications shall be made to mitigate this effect.

3.7.5 TIES AND CONTINUITY

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

As a minimum a positive connection for resisting a horizontal force shall be provided for each beam, girder, or truss to its support which shall have a minimum strength acting along the span of the member equal to 5 percent of the dead and live load reaction.

3.7.6 CONCRETE OR MASONRY WALL ANCHORAGE

Concrete and masonry walls shall be anchored to the roof and all floors which provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting a seismic lateral force, F_p, induced by the wall but not less than a force of $1000A_v$ (lbs) per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

3.7.7 ANCHORAGE OF NONSTRUCTURAL SYSTEMS

When required by Chapter 8, all portions or components of the building shall be anchored for the seismic force, F_p, prescribed therein.

3.7.8 COLLECTOR ELEMENTS

Collector elements shall be provided which are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces.

3.7.9 DIAPHRAGMS

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

Floor and roof diaphragms shall be designed to resist the seismic forces determined as follows.

3.7.9 Cont.

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

Diaphragms shall provide for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces, as prescribed in Sec. 3.7.5 or 8.2.2, into the diaphragm.

3.7.10 BEARING WALLS

Exterior and interior bearing walls and their anchorage shall be designed for a force of A_vW_c normal to the flat surface with a minimum of $0.1W_c$. Interconnection of dependent wall elements and connections to supporting framing systems shall have sufficient ductility or rotational capacity, or have sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

3.7.11 INVERTED PENDULUM-TYPE STRUCTURES

Inverted pendulum-type structures are structures where the seismic resisting system acts essentially as an isolated cantilever(s). Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Sec. 4.2 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

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 other horizontal components employing prestressing these effects may be satisfied by Formula 3-2a.

Sec. 3.8 DEFLECTION AND DRIFT LIMITS

All portions of the building shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact under total deflection, δ_x (as determined in Sec. 4.6.1), or modified deflection, $\bar{\delta}_x$ (as determined in Sec. 6.2.3), corresponding to the seismic design forces.

The design story drift, Δ, as determined in Sec. 4.6 or from Sec. 5.8 shall not exceed the allowable story drift, Δ_a, obtained from Table 3-C for any story.

TABLE 3-A

SOIL PROFILE COEFFICIENT

	Soil Profile Type		
	S_1	S_2	S_3
S	1.0	1.2	1.5

TABLE 3-B

RESPONSE MODIFICATION COEFFICIENTS[1]

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[7]	C_d[8]
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with shear panels	6½	4
	Shear walls		
	Reinforced concrete	4½	4
	Reinforced masonry	3½	3
	Braced frames	4	3½
	Unreinforced and partially reinforced masonry shear walls[6]	1¼	1¼
BUILDING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.	Light framed walls with Shear panels	7	4½
	Shear walls		
	Reinforced concrete	5½	5
	Reinforced masonry	4½	4
	Braced frames	5	4½
	Unreinforced and partially reinforced masonry shear walls[6]	1½	1½
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special moment frames		
	Steel[3]	8	5½
	Reinforced concrete[4]	7	6
	Ordinary moment frames		
	Steel[2]	4½	4
	Reinforced concrete[5]	2	2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided which shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their relative rigidities.	Shear walls		
	Reinforced concrete	8	6½
	Reinforced masonry	6½	5½
	Wood sheathed shear panels	8	5
	Braced frames	6	5
INVERTED PENDULUM STRUCTURES. Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Special Moment Frames		
	Structural steel[3]	2½	2½
	Reinforced concrete[4]	2½	2½
	Ordinary Moment Frames		
	Structural steel[2]	1¼	1¼

[1]These values are based on best judgement and data available at time of writing and need to be reviewed periodically.
[2]As defined in Sec. 10.4.1.
[3]As defined in Sec. 10.6
[4]As defined in Sec. 11.7.
[5]As defined in Sec. 11.4.1.
[6]Unreinforced masonry is not permitted for portions of buildings assigned to Category B. Unreinforced or partially reinforced masonry is not permitted for buildings assigned to Categories C and D; see Chapter 12.
[7]Coefficient for use in Formula 4-2, 4-3, and 5-3.
[8]Coefficient for use in Formula 4-9.

TABLE 3-C

ALLOWABLE STORY DRIFT Δ_a

	Seismic Hazard Exposure Group		
	III	II	I[1]
Δ_a	$0.010h_{sx}$	$0.015h_{sx}$	$0.015h_{sx}$

[1]Where there are no brittle-type finishes in buildings three stories or less in height, these limits may be increased one-third.

CHAPTER 4

EQUIVALENT LATERAL FORCE PROCEDURE

Sec. 4.1 GENERAL

The requirements of this Chapter shall control the seismic analysis of buildings as prescribed in Sec. 3.5.2 and 3.5.3.

Sec. 4.2 SEISMIC BASE SHEAR

The building , considered to be fixed at the base, shall be designed to resist the lateral seismic base shear, V, in the direction being analyzed as determined in accordance with the following formula:

$$V = C_s W \tag{4-1}$$

where

C_s = the seismic design coefficient.

W = the total gravity load of the building. W shall be taken equal to the total weight of the structure and applicable portions of other components including, but not limited to, the following:

1. Partitions and permanent equipment including operating contents.

2. For storage and warehouse structures, a minimum of 25 percent of the floor live load.

3. The effective snow load as defined in Sec. 2.1.

The value of C_s may be determined in accordance with Formula 4-2 or 4-3. Formula 4-2 requires calculation of the fundamental period of the building as specified in Sec. 4.2.2. If the period of the building is not calculated, C_s shall be determined using Formula 4-3 or 4-3a, as appropriate.

4.2.1 CALCULATION OF SEISMIC COEFFICIENT

When the period of the building is computed, the seismic coefficient C_s shall be determined in accordance with the following formula:

$$C_s = \frac{1.2 A_v S}{R T^{2/3}} \tag{4-2}$$

where

A_v = the coefficient representing Effective Peak Velocity-Related Acceleration from Sec. 1.4.1.

S = the coefficient for the soil profile characteristics of the site as given in Table 3-A.

R = the response modification factor as given in Table 3-B.

T = the fundamental period of the building as determined in Sec. 4.2.2.

C_s need not be taken as greater than the value given by Formula 4-3 or 4-3a.

4.2.1

The soil-structure interaction reduction as determined in Chapter 6 may be used.

For the design of a building where the period is not calculated the value of C_s shall be determined in accordance with the following formula:

$$C_s = 2.5\ A_a/R \tag{4-3}$$

where

A_a = the seismic coefficient representing the Effective Peak Acceleration as determined in Sec. 1.4.1.

EXCEPTION:
For Soil Profile Type S_3 in areas where $A_a \geq 0.30$, C_s shall be determined in accordance with the following formula:

$$C_s = 2A_a/R \tag{4-3a}$$

4.2.2 PERIOD DETERMINATION

The fundamental period of the building, T, in Formula 4-2 may be determined based on the properties of the seismic resisting system in the direction being analyzed and the use of established methods of mechanics assuming the base of the building to be fixed but shall not exceed 1.2 T_a. Alternatively the value of T may be taken equal to the approximate fundamental period of the building, T_a, used to establish a minimum seismic base shear for the building and determined in accordance with one of the following formulas:

For moment-resisting structures where the frames are not enclosed or adjoined by more rigid components tending to prevent the frames from deflecting when subjected to seismic forces:

$$T_a = C_T h_n^{3/4} \tag{4-4}$$

where

C_T = 0.035 for steel frames

C_T = 0.025 for concrete frames

h_n = the height in feet above the base to the highest level of the building.

For all other buildings:

$$T_a = \frac{0.05 h_n}{\sqrt{L}} \tag{4-5}$$

where

L = the overall length (in feet) of the building at the base in the direction under consideration.

4 Cont.

Sec. 4.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The lateral seismic shear force, F_x, induced at any level, shall be determined in accordance with the following formula:

$$F_x = C_{vx}V \tag{4-6}$$

where

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{4-6a}$$

k is an exponent related to the building period as follows:

For buildings having a period of 0.5 seconds or less, k = 1.

For buildings having a period of 2.5 seconds or more, k = 2.

For buildings having a period between 0.5 and 2.5 seconds, k may be taken as 2 or may be determined by linear interpolation between 1 and 2.

w_i, w_x = the portion of W located at or assigned to level i or x.

h_i, h_x = the height above the base to level i or x.

Sec. 4.4 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

The seismic shear force at any level, V_x, shall be determined in accordance with the following formula:

$$V_x = \sum_{i=x}^{n} F_i \tag{4-7}$$

The force, V_x, and the associated torsional forces shall be distributed to the various vertical components of the seismic resisting system in the story below level x with due consideration given to the relative stiffnesses of the vertical components and the diaphragm.

The design shall provide for the torsional moment M_t resulting from the location of the building masses plus the torsional moments M_{ta} caused by assumed displacement of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the building perpendicular to the direction of the applied forces.

Sec. 4.5 OVERTURNING

Every building shall be designed to resist overturning effects caused by the seismic forces determined in Sec. 4.3. At any level, the increment of overturning moment in the story under consideration shall be distributed to the various walls or frames in the same proportion as the distribution of the horizontal shears to those walls or frames.

The overturning moments shall be determined by the application of the prescribed forces as follows:

4.5 Cont.

$$M_x = \kappa \sum_{i=x}^{n} F_i(h_i - h_x) \tag{4-8}$$

where

κ = 1.0 for the top 10 stories

κ = 0.8 for the 20th story from the top and below

κ = a value between 1.0 and 0.8 determined by a straight line interpolation for stories between the 20th and 10th stories below the top

The foundations of buildings, except inverted pendulum structures, may be designed for the foundation overturning design moment, M_f, at the foundation-soil interface determined using Formula 4-8 with κ = 0.75 for all building heights. The resultant of the seismic forces and vertical loads at the foundation-soil interface shall not fall outside the middle one-half of the base of the component(s) resisting the overturning.

Sec. 4.6 DRIFT DETERMINATION AND P-DELTA EFFECTS

Story drifts and, where required, member forces and moments due to P-delta effects shall be determined in accordance with this Section.

4.6.1 STORY DRIFT DETERMINATION

The design story drift, Δ, shall be computed as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, shall be evaluated in accordance with the following formula:

$$\delta_x = C_d \, \delta_{xe} \tag{4-9}$$

where

C_d = the deflection amplification factor as given in Table 3-B

δ_{xe} = the deflections determined by an elastic analysis. The elastic analysis of the seismic resisting system shall be made using the prescribed seismic design forces (see Sec. 4.3) and considering the building fixed at the base.

For determining compliance with the story drift limitation of Sec. 3.8, the deflections δ_x may be calculated as above, but the seismic resisting system and the design forces corresponding to the fundamental period of the building, T, calculated without the limit specified in Sec. 4.2.2 may be used.

Where applicable Δ shall be increased by the incremental factor relating to the P-delta effects as determined in Sec. 4.6.2.

4.6.2 P-DELTA EFFECTS

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects need not be considered when the stability coefficient, θ, as determined in accordance with Formula 4-10, is equal to or less than 0.10.

4.6.2 Cont.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{4-10}$$

where

Δ = the design story drift.

V_x = the seismic shear force acting between level x and x-1.

h_{sx} = the story height below level x.

$P_x = \sum_{i=x}^{n} w_i$, total gravity load at and above level x

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

CHAPTER 5

MODAL ANALYSIS PROCEDURE

Sec. 5.1 GENERAL

The symbols used in this method of analysis have the same meaning as those for similar terms used in Chapter 4, with the subscript "m" denoting quantities in the m^{th} mode.

Sec. 5.2 MODELING

The building may be modeled as a system of masses lumped at the floor levels, with each mass having one degree of freedom, that of lateral displacement in the direction under consideration.

Sec. 5.3 MODES

The analysis shall include, for each of two mutually perpendicular axes, at least the lowest three modes of vibration or all modes of vibration with periods greater than 0.4 second, whichever is greater, except that for structures less than three stories in height, the number of modes shall equal the number of stories.

Sec. 5.4 PERIODS

The required periods and mode shapes of the building in the direction under consideration shall be calculated by established methods of mechanics for the fixed base condition using the masses and elastic stiffnesses of the seismic resisting system.

Sec. 5.5 MODAL BASE SHEAR

The portion of the base shear contributed by the the m^{th} mode, V_m, shall be determined in accordance with the following formula:

$$V_m = C_{sm}\overline{W}_m \tag{5-1}$$

where

C_{sm} = the modal seismic design coefficient determined below.

$\overline{W}_m$ = the effective modal gravity load determined in accordance with the following formula:

$$\overline{W}_m = \frac{\left[\sum_{i=1}^{n} w_i\phi_{im}\right]^2}{\sum_{i=1}^{n} w_i\phi_{im}^2} \tag{5-2}$$

where

ϕ_{im} = the displacement amplitude at the i^{th} level of the building when vibrating in its m^{th} mode.

The modal seismic design coefficient, C_{sm}, shall be determined in accordance with the following formula:

$$C_{sm} = \frac{1.2A_vS}{RT_m^{2/3}} \tag{5-3}$$

The value of C_{sm} need not exceed 2.5 A_a/R. For type S_3 soils in areas where the coefficient $A_a \geq 0.3$, C_{sm} need not exceed 2 A_a/R.

EXCEPTIONS:

1. For Soil Profile Type S_3 soils, C_{sm} for modes other than the fundamental mode which have periods less than 0.3 seconds may be determined in accordance with the following formula:

$$C_{sm} = \frac{A_a}{R}(0.8 + 4.0\ T_m) \qquad (5\text{-}3a)$$

2. For structures in which any T_m exceeds 4.0 seconds, the value of C_{sm} for that mode may be determined in accordance with the following formula:

$$C_{sm} = \frac{3A_vS}{RT_m^{4/3}} \qquad (5\text{-}3b)$$

The reduction due to soil-structure interaction may be used as determined in Sec. 6.3.

Sec. 5.6 MODAL FORCES, DEFLECTIONS AND DRIFTS

The modal force, F_{xm}, at each level shall be determined in accordance with the following formula:

$$F_{xm} = C_{vxm}V_m \qquad (5\text{-}4)$$

$$C_{vxm} = \frac{w_x\phi_{xm}}{\sum_{i=1}^{n} w_i\phi_{im}} \qquad (5\text{-}4a)$$

The modal deflection at each level, δ_{xm}, shall be determined in accordance with the following formula:

$$\delta_{xm} = C_d\ \delta_{xem} \qquad (5\text{-}5)$$

where

$$\delta_{xem} = \frac{g}{4\pi^2}\ \frac{T_m^2F_{xm}}{w_x} \qquad (5\text{-}6)$$

The modal drift in a story, Δ_m, shall be computed as the difference of the deflections, δ_{xm}, at the top and bottom of the story under consideration.

Sec. 5.7 MODAL STORY SHEARS AND MOMENTS

The story shears, story overturning moments, and the shear forces and overturning moments in walls and braced frames at each level due to the seismic forces determined from Formulas 5-4 and 5-5 shall be computed for each mode by linear static methods.

Sec. 5.8 DESIGN VALUES

The design value for base shear, each of the story shear, moment and drift quantities, and the deflection at each level shall be determined by combining their modal values, obtained from Sec. 5.6 and 5.7. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values. The design base shear, V_t, shall be compared with a base shear, $\overline{V}$, calculated using a period $T = 1.4T_a$ in Formula 4-2. Where V_t is less than $\overline{V}$, the design story shears, moments and drifts and floor deflections shall be multiplied by $\overline{V}/V_t$. The base shear need not exceed the values determined in accordance with Sec. 4.2.

Sec. 5.9 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

The provisions of Sec. 4.4 apply.

Sec. 5.10 FOUNDATION OVERTURNING

In the design of the foundation, the overturning moment at the foundation-soil interface may be reduced by 10 percent.

Sec. 5.11 P-DELTA EFFECTS

Using the story drifts and story shears determined in Sec. 5.8, the P-delta effects shall be determined in accordance with Sec. 4.6.2.

CHAPTER 6

SOIL-STRUCTURE INTERACTION

Sec. 6.1 GENERAL

The provisions set forth herein may be used to incorporate the effects of soil-structure interaction in the determination of the design earthquake forces and the corresponding displacements of the building. The use of these provisions will decrease the design values of the base shear, lateral forces and overturning moments, but may increase the computed values of the lateral displacements and of the secondary forces associated with the P-delta effects.

The provisions for use with the Equivalent Lateral Force Procedure are given in Sec. 6.2 and those for use with the Modal Analysis Procedure are given in Sec. 6.3.

Sec. 6.2 EQUIVALENT LATERAL FORCE PROCEDURE

The following provisions are supplementary to those presented in Chapter 4.

6.2.1 BASE SHEAR

The base shear, V, determined from Formulas 4-1 and 4-2 may be reduced to

$$\tilde{V} = V - \Delta V \tag{6-1}$$

to account for the effects of soil-structure interaction. The reduction, ΔV, shall be computed in accordance with the following formula:

$$\Delta V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\tilde{\beta}} \right)^{0.4} \right] \overline{W} \tag{6-2}$$

where

C_s = the seismic design coefficient computed from Formula 4-2 using the fundamental natural period of the fixed-base structure, T or T_a, as specified in Sec. 4.2.2.

$\tilde{C}_s$ = the value of C_s computed from Formula 4-2 using the fundamental natural period of the flexibly supported structure, $\tilde{T}$, defined in Sec. 6.2.1(A).

$\tilde{\beta}$ = the fraction of critical damping for the structure-foundation system, determined in Sec. 6.2.1(B).

$\overline{W}$ = the effective gravity load of the building, which shall be taken as 0.7 W, except that for buildings where the gravity load is concentrated at a single level, it shall be taken equal to W.

The reduced base shear, $\tilde{V}$, shall in no case be taken less than 0.7 V.

(A) EFFECTIVE BUILDING PERIOD. The effective period, $\tilde{T}$, shall be determined in accordance with the following formula:

$$\tilde{T} = T \sqrt{1 + \frac{\overline{k}}{K_y} \left(1 + \frac{K_y \overline{h}^2}{K_\theta} \right)} \tag{6-3}$$

where

$\bar{k}$ = the stiffness of the building when fixed at the base, defined by the following formula:

$$\bar{k} = 4\pi^2 \frac{\bar{W}}{gT^2} \qquad (6\text{-}4)$$

$\bar{h}$ = the effective height of the building. This shall be taken as 0.7 times the total height, h_n, except that for buildings where the gravity load is effectively concentrated at a single level, it shall be taken as the height to that level.

K_y = the lateral stiffness of the foundation, defined as the static horizontal force at the level of the foundation necessary to produce a unit deflection at that level, the force and the deflection being measured in the direction in which the structure is analyzed.

K_θ = the rocking stiffness of the foundation, defined as the static moment necessary to produce a unit average rotation of the foundation, the moment and rotation being measured in the direction in which the structure is analyzed.

g = the acceleration of gravity.

The foundation stiffnesses, K_y and K_θ, shall be computed by established principles of foundation mechanics (see Commentary) using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The average shear modulus, G, for the soils beneath the foundation at large strain levels, and the associated shear wave velocity, v_s, needed in these computations shall be determined from Table 6-A,

where

v_{so} = the average shear wave velocity for the soils beneath the foundation at small strain levels (10^{-3} percent or less).

G_o = $\gamma v_{so}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels.

γ = the average unit weight of the soils.

TABLE 6-A

VALUES OF G/G_o AND v_s/v_{so}

Ground Acceleration Coefficient, A_v	≤ 0.10	0.15	0.20	≥ 0.30
Value of G/G_o	0.81	0.64	0.49	0.42
Value of v_s/v_{so}	0.9	0.8	0.7	0.65

Alternatively, for buildings supported on mat foundations that rest at or near the ground surface, or are embedded in such a way that the side wall contact with the soil cannot be considered to remain effective during the design ground motion, the effective period of the building may be determined in accordance with the following formula:

$$\tilde{T} = T\sqrt{1 + 25\alpha \frac{r_a \bar{h}}{v_s^2 T^2}\left(1 + 1.12\frac{r_a \bar{h}^2}{r_m^3}\right)} \qquad (6\text{-}5)$$

where

α = the relative weight density of the structure and the soil, defined by

$$\alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \qquad (6\text{-}6)$$

r_a and r_m = characteristic foundation lengths, defined by:

$$r_a = \sqrt{\frac{A_o}{\pi}} \qquad (6\text{-}7)$$

$$r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad (6\text{-}8)$$

A_o = the area of the foundation.

I_o = the static moment of the foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed.

(B) EFFECTIVE DAMPING. The effective damping factor for the structure-foundation system, $\tilde{\beta}$, shall be computed from the following formula:

$$\tilde{\beta} = \beta_o + \frac{0.05}{(\tilde{T}/T)^3} \qquad (6\text{-}9)$$

where

β_o = the foundation damping factor as specified in Figure 6-1.

The values of β_o corresponding to $A_v = 0.15$ in this figure shall be determined by averaging the results obtained from the solid lines and the dashed lines.

The quantity r in Figure 6-1 is a characteristic foundation length, which shall be determined in accordance with the following formulas:

For $\frac{\bar{h}}{L_o} \leq 0.5$ $\qquad r = r_a = \sqrt{\frac{A_o}{\pi}}$

For $\frac{\bar{h}}{L_o} \geq 1$ $\qquad r = r_m = \sqrt[4]{\frac{4I_o}{\pi}}$

6.2.1(B) Cont.

where

L_o = the overall length of the side of the foundation in the direction being analyzed.

A_o = the area of the load carrying foundation.

I_o = the static moment of inertia of the load carrying foundation.

For intermediate values of $\overline{h}/L_o$, the value of r shall be determined by linear interpolation.

EXCEPTION:
For buildings supported on point bearing piles and in all other cases where the foundation soil consists of a soft stratum of reasonably uniform properties underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, the factor β_o in Formula 6-9 shall be replaced by:

$$\beta_o' = \left(\frac{4D_s}{v_s\tilde{T}}\right)^2 \beta_o \qquad (6\text{-}10)$$

if

$$\frac{4D_s}{v_s\tilde{T}} < 1$$

where

D_s = the total depth of the stratum.

The value of $\tilde{\beta}$ computed from Formula 6-9, both with or without the adjustment represented by Formula 6-10, shall in no case be taken as less than $\tilde{\beta} = 0.05$.

6.2.2 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The distribution over the height of the building of the reduced total seismic force, $\tilde{V}$, shall be considered to be the same as for the building without interaction.

6.2.3 OTHER EFFECTS

The modified story shears, overturning moments and torsional effects about a vertical axis shall be determined as for structures without interaction using the reduced lateral forces.

The modified deflections, $\tilde{\delta}_x$, shall be determined in accordance with the following formula:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V}\left[\frac{M_o h_x}{K_\theta} + \delta_x\right] \qquad (6\text{-}11)$$

where

M_o = the overturning moment at the base determined in accordance with Sec. 4.5 using the unmodified seismic forces. This moment should not include the reduction permitted in the design of the foundation.

6.2.3 Cont.

h_x = the height above the base to the level under consideration.

δ_x = the deflections of the fixed base structure as determined in Sec. 4.6.1 using the unmodified seismic forces.

The modified story drifts and P-delta effects shall be evaluated in accordance with the provisions of Sec. 4.6 using the modified story shears and deflections determined in this Section.

Sec. 6.3 MODAL ANALYSIS PROCEDURE

The following provisions are supplementary to those presented in Chapter 5.

6.3.1 MODAL BASE SHEARS

The base shear corresponding to the fundamental mode of vibration, V_1, may be reduced to

$$\tilde{V}_1 = V_1 - \Delta V_1 \qquad (6\text{-}12)$$

to account for the effects of soil-structure interaction. The reduction, ΔV_1, shall be computed in accordance with Formula 6-2,

where

$\overline{W}$ shall be taken equal to the gravity load $\overline{W}_1$ defined by Formula 5-2.

C_s shall be computed from Formula 5-3 using the fundamental period of the fixed-base building, T_1.

$\tilde{C}_s$ shall be computed from Formula 5-3 using the fundamental period of the elastically supported building $\tilde{T}_1$.

The period $\tilde{T}_1$ shall be determined from Formula 6-3, or from Formula 6-5 when applicable, taking $T = T_1$, evaluating $\overline{k}$ from Formula 6-4 with $\overline{W} = \overline{W}_1$, and computing $\overline{h}$ in accordance with the formula:

$$\overline{h} = \frac{\sum_{i=1}^{n} w_i \phi_{i1} h_i}{\sum_{i=1}^{n} w_i \phi_{i1}} \qquad (6\text{-}13)$$

The above designated values of $\overline{W}$, $\overline{h}$, T, and $\tilde{T}$ shall also be used to evaluate the factor α from Formula 6-6 and the factor β_o from Figure 6-1.

No reduction shall be made in the shear components contributed by the higher modes of vibration.

The reduced base shear, $\tilde{V}_1$, shall in no case be taken less than 0.7 V_1.

6.3.2 OTHER MODAL EFFECTS

The modified modal seismic forces, story shears and overturning moments shall be determined as for buildings without interaction using the modified base shear, $\tilde{V}_1$, instead of V_1.

6.3.2 Cont.

The modified modal deflections, $\tilde{\delta}_{xm}$, shall be determined in accordance with the following formulas:

$$\tilde{\delta}_{x1} = \frac{\tilde{V}_1}{V_1}\left[\frac{M_{o1}h_x}{K_\theta} + \delta_{x1}\right] \tag{6-14}$$

and

$$\tilde{\delta}_{xm} = \delta_{xm} \qquad \text{for } m = 2, 3, \ldots \tag{6-15}$$

where

M_{o1} = the overturning base moment for the fundamental mode of the fixed-base building, as determined in Sec. 5.7 using the unmodified modal base shear V_1.

δ_{xm} = the modal deflections at level x of the fixed-base building, as determined in Sec. 5.6 using the unmodified modal shears, V_m.

The modified modal drift in a story, $\tilde{\Delta}_m$, shall be computed as the difference of the deflections, $\tilde{\delta}_{xm}$, at the top and bottom of the story under consideration.

6.3.3 DESIGN VALUES

The design values of the modified shears, moments, deflections, and story drifts shall be determined as for structures without interaction by taking the square root of the sum of the squares of the respective modal contributions. In the design of the foundation, the overturning moment at the foundation-soil interface determined in this manner may be reduced by 10 percent, as for structures without interaction.

The effects of torsion about a vertical axis shall be evaluated in accordance with the provisions of Sec. 4.4 and the P-delta effects shall be evaluated in accordance with the provisions of Sec. 4.6.2, using the story shears and drifts determined in Sec. 6.3.2.

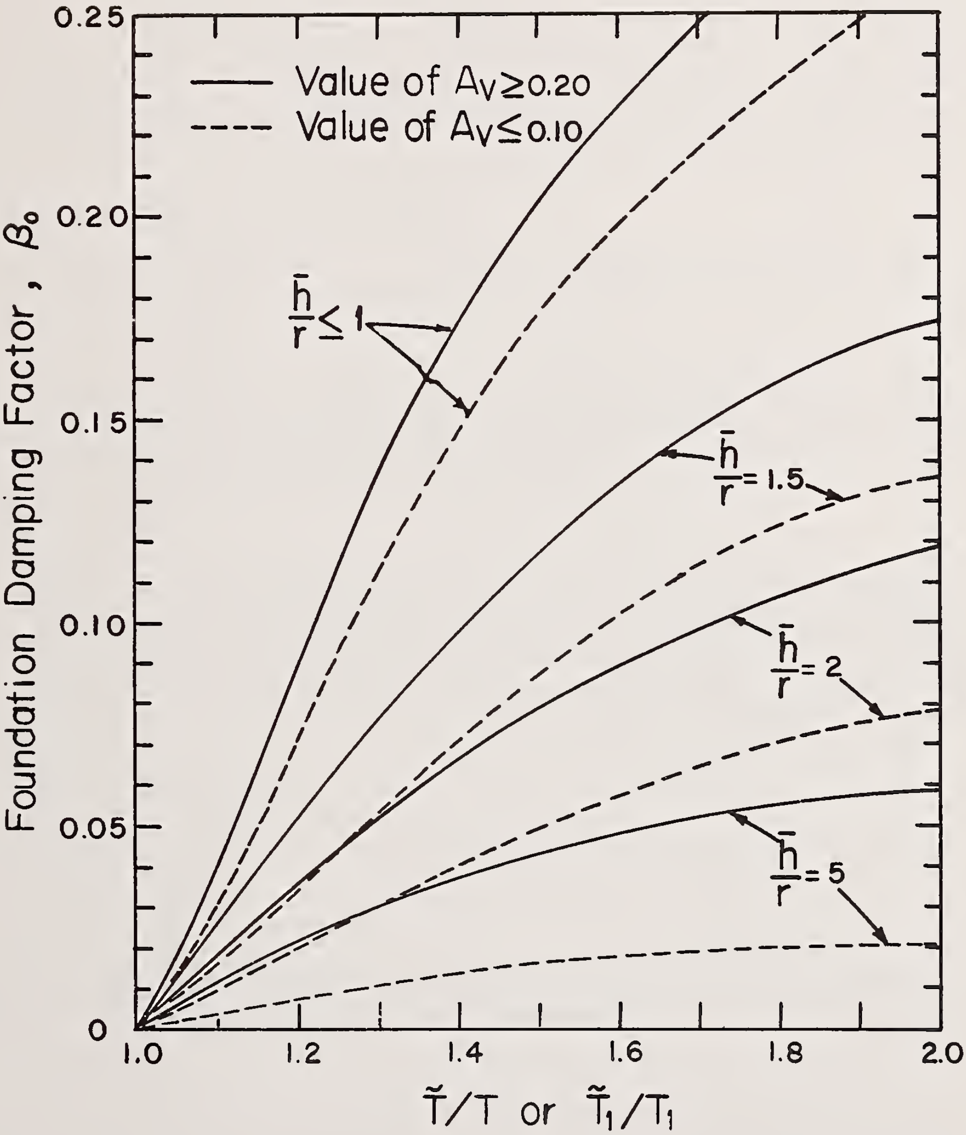


FIGURE 6-1 FOUNDATION DAMPING FACTOR

CHAPTER 7

FOUNDATION DESIGN REQUIREMENTS

Sec. 7.1 GENERAL

This Chapter includes only those foundation requirements that are specifically related to seismic resistant construction. It assumes compliance with all the basic requirements necessary to provide support for vertical loads and lateral loads other than those due to earthquake motions. These include, but are not limited to, provisions for the extent of investigation, fills, slope stability, bearing and lateral soil pressures, reports, drainage, settlement control, and pile requirements and capacities.

Sec. 7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS

For the seismic forces prescribed in Chapters 1 through 6, the resisting capacities of the foundations shall meet the requirements of this Chapter.

7.2.1 STRUCTURAL MATERIALS

The strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall be as determined in Chapter 9, 10, 11, or 12. The strength of foundation components shall not be less than that required for forces acting without seismic forces.

7.2.2 SOIL CAPACITIES

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

Sec. 7.3 SEISMIC PERFORMANCE CATEGORY A

Any construction meeting the requirements of Sec. 7.1 and 7.2 may be used for buildings classified as Category A.

Sec. 7.4 SEISMIC PERFORMANCE CATEGORY B

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

7.4.1 INVESTIGATION

The Regulatory Agency may require the submission of a written report which shall include, in addition to the requirements of Sec. 7.1 and the evaluations required in Sec. 7.2.2, the results of an investigation to determine the potential hazards due to (a) slope instability, (b) liquefaction, and (c) surface rupture due to faulting or lurching, all as a result of earthquake motions.

7.4.2 POLE-TYPE STRUCTURES

Construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth may be used to resist both axial and lateral loads. The depth of embedment required for posts or poles to resist seismic loads shall be determined by means of the design criteria established in the foundation investigation report.

7.4.3 FOUNDATION TIES

Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger pile cap or column load unless it can be demonstrated that equivalent restraint can be provided by other approved means.

7.4.4 SPECIAL PILE REQUIREMENTS

The following special requirements for concrete or composite concrete and steel piles are in addition to requirements for piles in the code administered by the Regulatory Agency.

The piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in Chapter 11. For deformed bars the development length is for compression without reduction in length for excess area.

(A) UNCASED CONCRETE PILES. Reinforcing steel shall be provided in the top portion of uncased cast-in-place concrete piles or caissons for a distance of ten pile diameters, with a minimum steel ratio of 0.0025 with a minimum of four No. 5 bars. Ties (or equivalent spirals) shall be provided at 16-bar diameter spacing (maximum spacing) with a maximum spacing of 4 inches in the top 2 feet.

(B) METAL-CASED CONCRETE PILES. Reinforcing steel shall be provided for metal-cased concrete piles in the upper one-third of the pile length (8-foot minimum) with a mininum steel ratio of 0.005 with spiral ties of 1/4-inch diameter minimum at 9-inch maximum pitch or equivalent ties. For the top 2 feet below the pile cap reinforcing, the pitch shall be 3 inches maximum.

(C) STEEL PIPE PILES. Reinforcement equal to one percent of the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment.

(D) PRECAST PILES. Longitudinal reinforcing steel shall be provided for precast concrete piles with a minimum steel ratio of 0.01.

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

Sec. 7.5 SEISMIC PERFORMANCE CATEGORY C

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

7.5.1 INVESTIGATION

The Regulatory Agency may require the submission of a written report which shall include, in addition to the requirements of Sec. 7.4.1, the determination of lateral pressures on basement and retaining walls due to earthquake motions.

7.5.2 FOUNDATION TIES

Individual spread footings shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by other approved means.

7.5 Cont.

7.5.3 SPECIAL PILE REQUIREMENTS

The following special requirements shall apply:

(A) UNCASED CONCRETE PILES. Reinforcing steel shall be provided for uncased cast-in-place concrete piles, drilled piers or caissons with a minimum steel ratio of 0.005 with a minimum of four No. 6 bars. Ties shall be provided at eight-bar-diameter spacing with a maximum spacing of 3 inches in the top 4 feet. Ties shall be a minimum of No. 3 bars for up to 20-inch-diameter piles and No. 4 bars for piles of larger diameter.

(B) METAL-CASED CONCRETE PILES. Reinforcing steel shall be provided for metal-cased concrete piles for the full length of the pile. The upper two-thirds of the pile shall have a minimum of 4 bars with a minimum steel ratio of 0.0075 with a minimum of 1/4-inch diameter spiral ties at 9-inch maximum pitch. At the top 4 feet, the pitch shall be 3 inches maximum.

(C) PRECAST CONCRETE PILES. Ties in precast concrete piles shall conform to the requirements of Sec. 11.6.2 for the top half of the pile. Precast concrete piles shall not be used to resist flexure caused by earthquake motions unless it can be shown that they will be stressed to below the elastic limit under the maximum soil deformations that would occur during an earthquake.

(D) STEEL PILES. The connection between the pile cap and steel piles or unfilled steel pipe piles shall be designed for a tensile force equal to ten percent of the pile compression capacity.

Sec. 7.6 SEISMIC PERFORMANCE CATEGORY D

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

Sec. 7.6.1 SPECIAL PILE LIMITATIONS

Precast-prestressed piles shall not be used to resist flexure caused by earthquake motions.

CHAPTER 8

ARCHITECTURAL, MECHANICAL AND ELECTRICAL COMPONENTS AND SYSTEMS

Sec. 8.1 GENERAL REQUIREMENTS

The requirements of this Chapter establish minimum design levels for architectural, mechanical, and electrical systems and components recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical and electrical components. All architectural, mechanical, and electrical systems and components in buildings and portions thereof shall be designed and constructed to resist seismic forces determined in accordance with this Chapter.

EXCEPTIONS:

1. Those systems or components designated in Table 8-B or 8-C for L performance level which are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with a Seismicity Index of 1 or 2 or which are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with a Seismicity Index of 1 are not subject to the provisions of this Chapter.
2. Where alterations or repairs are made the forces on systems or components in existing buildings may be modified in accordance with the provisions of Sec. 13.3.

Seismic Hazard Exposure Groups are determined in Sec. 1.4. Mixed Occupancy requirements are provided in that section.

The seismic force on any component shall be applied at the center of gravity of the component and shall be assumed to act in any horizontal direction. For vertical forces on mechanical and electrical components, see Table 8-C, Footnote 2.

8.1.1 INTERRELATIONSHIP OF COMPONENTS

The interrelationship of systems or components and their effect on each other shall be considered so that the failure of an architectural, mechanical, or electrical system or component of one performance level shall not cause an architectural, mechanical, or electrical system or component of higher level to fail in its performance requirements.

The effect on the response of the structural system and deformational compatibility of architectural, electrical, and mechanical systems or components shall be considered where there is interaction of these systems or components with the structural system.

8.1.2 CONNECTIONS AND ATTACHMENTS

Architectural, electrical, and mechanical systems and components required to be designed to resist seismic forces shall be attached so that the forces are ultimately transferred to the structure of the building. The attachment shall be designed to resist the prescribed forces.

Friction due to gravity shall not be considered in evaluating the required resistance to seismic forces.

The design documents shall include sufficient information relating to the attachments to verify compliance with the requirements of this Chapter.

8.1.3 PERFORMANCE CRITERIA

The performance criteria for architectural, mechanical, and electrical components and systems are listed in Table 8-A for use in Formulas 8-1 and 8-2 and Tables 8-B and 8-C.

8.1.3 Cont.

These components and systems shall be designed to meet the performance characteristic levels established in Tables 8-B and 8-C.

Sec. 8.2 ARCHITECTURAL DESIGN REQUIREMENTS

8.2.1 GENERAL

Systems or components listed in Table 8-B and their attachments shall be designed and detailed in accordance with the requirements of this Chapter. The designs or criteria for systems or components shall be included as part of the design documents.

8.2.2 FORCES

Architectural systems and components and their attachments shall be designed to resist seismic forces determined in accordance with the following formula:

$$F_p = A_v C_c P W_c \qquad (8\text{-}1)$$

where

F_p = The seismic force applied to a component of a building or equipment at its center of gravity.

C_c = The seismic coefficient for components of architectural systems as given in Table 8-B (dimensionless).

W_c = The weight of a component of a building or equipment.

A_v = The seismic coefficient representing the Effective Peak Velocity-Related Acceleration as determined in Sec. 1.4.

P = Performance criteria factor as given in Table 8-A (dimensionless).

EXCEPTIONS:
When positive and negative wind loads exceed F_p for nonbearing exterior walls, these loads shall govern the design. Similarly, when the code horizontal loads exceed F_p for interior partitions, these loads shall govern the design.

8.2.3 EXTERIOR WALL PANEL ATTACHMENT

Attachment of exterior wall panels to the building seismic resisting system shall have sufficient ductility and provide rotational capacity needed to accomodate the design story drift determined in accordance with Sec. 4.6.1.

8.2.4 COMPONENT DEFORMATION

Provisions shall be made in the architectural system or component for the design story drift Δ as determined in Sec. 4.6.1. Provision shall be made for vertical deflection due to joint rotation of cantilever members.

EXCEPTION:
Components assigned an L performance factor in Table 8-B may provide for a design story drift of $\Delta/2$.

8.2 Cont.

8.2.5 OUT-OF-PLANE BENDING

Transverse or out-of-plane bending or deformation of a component or system composed of basically brittle materials which are subject to forces as determined in Formula 8-1 shall not exceed the deflection capability of the material.

Sec. 8.3 MECHANICAL AND ELECTRICAL DESIGN REQUIREMENTS

8.3.1 GENERAL

Systems or components listed in Table 8-C and their attachments shall be designed and detailed in accordance with the requirements of this Chapter. The designs or criteria for systems or components shall be included as part of the design documents.

An analysis of a component supporting mechanism based on established principles of structural dynamics may be performed to justify reducing the forces determined in Sec. 8.3.2.

Combined states of stress, such as tension and shear in anchor bolts, shall be investigated in accordance with established principles of mechanics.

8.3.2 FORCES

Mechanical and electrical components and their attachments shall be designed for seismic forces determined in accordance with the following formula:

$$F_p = A_v\, C_c\, P\, a_c\, a_x\, W_c \qquad (8\text{-}2)$$

where

F_p, A_v, P, and W_c are as defined previously in Sec. 8.2.2.

C_c = The seismic coefficient for components of mechanical or electrical systems as given in Table 8-C (dimensionless).

a_c = The amplification factor related to the response of a system or component as affected by the type of attachment, determined in Sec. 8.3.2(A).

a_x = The amplification factor at level x related to the variation of the response in height of the building.

The amplification factor, a_x, shall be determined in accordance with the following formula:

$$a_x = 1.0 + (h_x/h_n) \qquad (8\text{-}3)$$

where h_x = The height above the base to level x

h_n = The height above the base to level n

(A) ATTACHMENT AMPLIFICATION. The attachment amplification factor, a_c, shall be determined as follows:

For fixed or direct attachment to buildings: $a_c = 1.$

For resilient mounting system:

with seismic activated restraining device $a_c = 1.$

with elastic restraining device:

if $T_c/T < 0.6$ or $T_c/T > 1.4$ $\quad a_c = 1.$

if $T_c/T \geq 0.6$ or ≤ 1.4 $\quad a_c = 2$ minimum*.

if mounted on the ground or on a slab in direct contact with the ground $\quad a_c = 2.$

The value of the fundamental period, T, shall be the value used in the design of the building as determined in accordance with Sec. 4.2 or Sec. 5.4.

The fundamental period of the component and its attachment, T_c, shall be determined in accordance with the following formula:

$$T_c = 0.32 \sqrt{W_c/K} \tag{8-4}$$

where

K = The stiffness of the equipment support attachment determined in terms of load per unit deflection of the center of gravity (lbs./in.) as follows:

For stable resilient attachments, K = spring constant.

For other resilient attachments, K = slope of the load/deflection curve at the point of loading.

In lieu of Formula 8-4, properly substantiated values for T_c derived using experimental data or any generally accepted analytical procedure may be used.

8.3.3 ATTACHMENT DESIGN

Fixed or direct attachments shall be designed for the forces determined in Sec. 8.3.2 and in conformance with Chapters 9, 10, 11, or 12 for the materials comprising the attachment.

Resilient mounting devices shall be of the stable type. Restraining devices shall be provided to limit the horizontal and vertical motion, to inhibit the forces from forcing the resilient mounting system into resonance, and to prevent overturning. Elastic restraining devices shall be designed based upon the forces obtained from Formula 8-2 or in accordance with the dynamic properties of the component and the structure to which it is attached. Horizontal and vertical elastic restraining devices shall be designed to decelerate the component or system on contact at a rate which will not generate forces in excess of those calculated from Formula 8-2.

8.3.4 COMPONENT DESIGN

When the direct attachment method is to be used for components with performance characteristic levels of S or G in areas with Seismicity Index 3 or 4, the designer shall require certification from the manufacturer that the components will not sustain damage if subjected to forces equivalent to those resulting from Formula 8-2.

*See Sec. 8.3 of Commentary.

8.3.4 Cont.

When resilient mounting systems are used for components with performance criteria levels S or G both the mounting systems and the components shall require the certification stated above. Such systems shall be of the stable type.

Testing and certification shall be in accordance with the requirements of Sec. 1.6.3.

8.3.5 UTILITY AND SERVICE INTERFACES

The utility or service interface of all gas, high-temperature energy and electrical supply to buildings housing Seismic Hazard Exposure Groups II and III and located in areas having a Seismicity Index of 3 or 4 shall be provided with shutoff devices located at the building side of the interface. Such shutoff devices shall be activated either by a failure within a system being supplied or by a mechanism which will operate when the ground motion exceeds 0.5 A_a times the acceleration of gravity.

TABLE 8-A

PERFORMANCE CRITERIA

Designation[1]	Performance Characteristic Level	P
S	Superior	1.5
G	Good	1.0
L	Low	0.5

[1]See Tables 8-B and 8-C.

TABLE 8-B

SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS
REQUIRED FOR ARCHITECTURAL SYSTEMS OR COMPONENTS
(See Table 8-A for S, G and L Designations)

Architectural Components	C_c Factor	Required Performance Characteristic Levels Seismic Hazard Exposure Group III	II	I
Appendages				
Exterior Nonbearing Walls	.9	S	G^2	L^4
Wall Attachments	3.0	S	G^2	L^4
Veneers	3.0	G	G^1	L
Roofing Units	.6	G	G^2	NR
Containers and Miscellaneous Components (free standing)	1.5	G	G	NR
Partitions				
Stairs and Shafts	1.5	S	G^3	G
Elevators and Shafts	1.5	S	L^3	L^5
Vertical Shafts	.9	S	L^3	L^6
Horizontal Exits including Ceilings	.9	S	S	G
Public Corridors	.9	S	G	L
Private Corridors	.6	S	L	NR
Full-height Area Separation Partitions	.9	S	G	G
Full-height Other Partitions	.6	S	L	L
Partial-height Partitions	.6	G	L	NR
Structural Fireproofing	.9	S	G^3	L^6
Ceilings - Fire-rated Membrane	.9	S	G^3	G
- Nonfire-rated Membrane	.6	G	G	L
Architectural Equipment - Ceiling, Wall, or Floor Mounted	.9	S	G	L

NR = Not required.

[1]May be reduced one performance level if the area facing the exterior wall is nominally inaccessible for a distance of 10 feet plus one foot for each floor of height.
[2]May be reduced one performance level if the area facing the exterior wall is nominally inaccessible for a distance of 10 feet and building is only one story.
[3]Shall be raised one performance level if building is more than four stories or 40 feet in height.
[4]Shall be raised one performance level if building is in an urban area.
[5]May be reduced to NR if building is less than 40 feet in height.
[6]Shall be raised one performance level for an occupancy containing flammable gases, liquids, or dust.

TABLE 8-C

SEISMIC COEFFICIENT (C_c) AND PERFORMANCE CHARACTERISTIC LEVELS REQUIRED FOR MECHANICAL/ELECTRICAL COMPONENTS
(See Table 8-A for S, G and L Designations)

Mechanical/Electrical Components[1]	C_c Factor[2]	Required Performance Characteristic Levels Seismic Hazard Exposure Group III	II	I
Emergency Electrical Systems (code required) Fire and Smoke Detection System (code required) Fire Supperssion Systems (code required) Life Safety System Components	2.00	S	S	S
Boilers, Furnaces, Incinerators, Water Heaters, and Other Equipment Using Combustible Energy Sources or High Temperature Energy Sources, Chimneys, Flues, Smokestacks, and Vents Communication Systems Electrical Bus Ducts and Primary Cable Systems Electrical Motor Control Centers, Motor Control Devices, Switchgear, Transformers, and Unit Substations Reciprocating or Rotating Equipment Tanks, Heat Exchangers, and Pressure Vessels Utility and Service Interfaces	2.00	S	G	L
Machinery (Manufacturing and Process)	.67	S	G	L
Lighting Fixtures	.67[3]	S	G	L
Ducts and Piping Distribution Systems				
- Resiliently Supported	2.00	S	G	NR
- Rigidly Supported	.67[4]	S	G	NR
Electrical Panelboards and Dimmers	.67	S	G	NR
Conveyor Systems (non-personnel)	.67	S	NR	NR

NR = Not Required.

[1]Where mechanical or electrical components are not specifically listed in Table 8-C, the designer shall select a similarly listed component, subject to the approval of the authority having jurisdiction, and shall base the design on the performance and C_c values for the similar component.
[2]C_c values listed are for horizontal forces. C_c values for vertical forces shall be taken as 1/3 of the horizontal values.
[3]Hanging- or swinging-type fixtures shall use a C_c value of 1.5 and shall have a safety cable attached to the structure and the fixture at each support point capable of supporting 4 times the vertical load.
[4]Seismic restraints may be omitted from the following installations:

a. Gas piping less than 1-inch inside diameter.
b. Piping in boiler and mechanical rooms less than 1-1/4 inches inside diameter.
c. All other piping less than 2-1/2 inches inside diameter.
d. All electrical conduit less than 2-1/2 inches inside diameter.
e. All rectangular air-handling ducts less than 6 square feet in cross-sectional area.
f. All round air-handling ducts less than 28 inches in diameter.
g. All piping suspended by individual hangers 12 inches or less in length from the top of the pipe to the bottom of the support for the hanger.
h. All ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

CHAPTER 9

WOOD

Sec. 9.1 REFERENCE DOCUMENTS

The quality, testing, design and construction of members and their fastenings in wood systems which resist seismic forces shall conform to the requirements of the reference documents listed in this Section except as modified by the provisions of this Chapter.

Ref. 9.1	National Design Specification for Stress Grade Lumber and Its Fastenings	NDS (1977)
Ref. 9.2	American Softwood Lumber Standard	PS 20
Ref. 9.3	Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber	ASTM D245 (1974)
Ref. 9.4	Methods for Establishing Clear Wood Strength Values	ASTM D2555 (1976)
Ref. 9.5	Softwood Plywood - Construction and Industrial	PS 1 (1974)
Ref. 9.6	Particle Board	CS236-66
Ref. 9.7	Preservative Treatment by Pressure Process	AWPA C1, C2, C3, and C29 (1976), C24 (1974)
Ref. 9.8	Structural Glued-Laminated Timber	PS 56 (1973)
Ref. 9.9	Structural Glued-Laminated Timber	AITC (1976)
Ref. 9.10	Wood Poles	ANSI 05.1 (1972)
Ref. 9.11	Round Timber Piles	ASTM D25 (1970)
Ref. 9.12	One- and Two-family Dwelling Codes	
	International Conference of Building Officials	1975
	Building Officials and Code Admintrators	1975
	Southern Building Code Congress	1975
Ref. 9.13	Gypsum Wallboard	ASTM C36-64
Ref. 9.14	Fiberboard, Insulating	ASTM D2277-64T

Sec. 9.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor ϕ and 2.0 times the working stresses permitted in the reference documents and in this Chapter.

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

Bending, bearing, and axial compression or tension in wood members	$\phi = 1.0$
Nails in shear in plywood diaphragms when calculating strength by principles of mechanics	
In Group III species members	$\phi = 0.82$
In Group IV species members	$\phi = 0.65$
Shear on diaphragms and shear walls as given in this Chapter	$\phi = 0.75$
Shear on carriage bolts not having washers under the heads	$\phi = 0.67$
Lag screws and wood screws in joints with not more than 4 screws	$\phi = 0.90$
Lag screws and wood screws in joints with more than 4 screws	$\phi = 3.6/N$
Bolts and other timber connections	$\phi = 1.0$

Sec. 9.3 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be constructed using any of the materials and procedures permitted in the reference documents and this Chapter except as limited in this Section.

9.3.1 BRACING REQUIREMENTS

All wood frame buildings three stories in height shall have solid sheathing of one of the materials specified in Sec. 9.7.3 applied for the full height over not less than 25 percent of the length of each exterior wall in the first story.

Sec. 9.4 SEISMIC PERFORMANCE CATEGORY B

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

9.4.1 DETAILING REQUIREMENTS

The construction shall comply with the requirements given below.

(A) ANCHORAGE OF CONCRETE OR MASONRY WALLS. The diaphragm sheathing shall not be used for providing ties and splices required in Sec. 3.7.5 and 3.7.6.

(B) LAG SCREWS. Washers shall be provided under the heads of lag screws that would otherwise bear on wood.

(C) ECCENTRIC JOINTS. The 50 percent increase permitted for allowable working stresses in Sec. 208B of Ref. 9.1 shall be limited to joints where all parts of the joint are located within five times the depth of the member from the end of the piece.

Sec. 9.5 SEISMIC PERFORMANCE CATEGORY C

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

9.5.1 MATERIAL LIMITATIONS

The limitations on materials used in Category C construction are given below.

(A) PLYWOOD. Where plywood is used structurally as covering on the exterior of outside walls, it shall be of the exterior type. Where used elsewhere structurally, it shall be bonded by intermediate or exterior glue.

9.5 Cont.

9.5.2 FRAMING SYSTEMS

The limitations on framing systems which may be used in Category C are given below.

(A) DIAPHRAGMS. Wood diaphragms shall not be used to resist torsional forces induced by concrete or masonry construction in structures over two stories in height.

(B) SHEAR WALLS. The use of walls sheathed with gypsum sheathing, particle board, gypsum wall board, or wire lath and cement plaster as shear walls for resisting seismic forces shall be limited to one-story buildings or the top story of buildings two stories or more in height. Fiberboard sheathed shear walls shall not be used as part of the seismic force resisting system.

(C) CONVENTIONAL LIGHT FRAME CONSTRUCTION. Buildings over one story in height of conventional light frame construction shall have solid sheathing of one of the materials specified in Sec. 9.7.3(A) or (B) applied for the full height over at least 40 percent of the length of the building at each exterior wall of the stories below the top story.

9.5.3 DETAILING REQUIREMENTS

Special details for Category C construction are given below.

(A) NAILS. Common wire nails driven parallel to the grain of the wood shall not be used to resist loads greater than 50 percent of working stress values permitted in Ref. 9.1 for normal duration of loading for nails driven perpendicular to the grain.

Connections using multiple nails driven perpendicular to the grain and used to resist loads in withdrawal shall use the capacity reduction factors given for lag screws and wood screws.

(B) PLYWOOD SHEAR PANELS. Plywood used for shear panels which are a part of the seismic resisting system shall be applied directly to the framing members, except that plywood may be used as a diaphragm when nailed over solid lumber planking or laminated decks.

Sec. 9.6 SEISMIC PERFORMANCE CATEGORY D

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

9.6.1 MATERIAL LIMITATIONS

Walls sheathed with gypsum sheathing, particle board, gypsum wall board, fiberboard, or wire lath and cement plaster shall not be used as part of the seismic resisting system.

9.6.2 FRAMING SYSTEMS

Unblocked plywood diaphragms shall not be used as part of the seismic resisting system.

9.6.3 DIAPHRAGM LIMITATIONS

The allowable working stress shear for vertical plywood shear walls, used to resist horizontal forces in buildings with masonry or reinforced concrete walls, shall be one-half of the allowable values set forth in Table 9-2.

Sec. 9.7 CONVENTIONAL LIGHT TIMBER CONSTRUCTION

Wood frame buildings which require no engineering analysis of the seismic loading effects, in accordance with Sec. 1.3.1, shall be subject to the design regulations enforced by the Regulatory Agency for general wood-frame and light-frame construction except as modified by the provisions of this Section.

9.7.1 WALL FRAMING AND CONNECTIONS

The following wall framing and connection details shall apply as a minimum.

(A) ANCHOR BOLTS. Foundation sill anchor bolts at least 1/2 inch in diameter shall be provided at not over 4 feet on center. Anchor bolts shall have a minimum embedment of 7 diameters.

(B) TOP PLATES. Stud walls shall be capped with double-top plates installed to provide overlapping at corners and intersections. End joints in double-top plates shall be offset at least 48 inches.

(C) BOTTOM PLATES. Studs shall have full bearing on a plate or sill of not less than 2-inch nominal thickness and having a width at least equal to the width of the stud.

9.7.2 WALL SHEATHING REQUIREMENTS

All exterior walls and main interior partitions shall be effectively and thoroughly braced by one of the types of sheathing described in Sec. 9.7.3 at each end of the wall or partition, or as near thereto as possible, and at not over 25-foot intervals between the ends. To be considered effective as bracing, the sheathing shall be at least 48 inches in width covering three 16-inch stud spaces or two 24-inch stud spaces. All vertical joints of panel sheathing shall occur over studs and all horizontal joints shall occur over blocking at least equal in size to the studs. All framing in connection with sheathing used for bracing shall not be less than 2-inch nominal thickness.

9.7.3 ACCEPTABLE TYPES OF WALL SHEATHING

Sheathing used for bracing shall conform to one of the following types of construction:

(A) DIAGONAL BOARDS. Wood boards of 5/8-inch minimum net thickness applied diagonally on studs spaced not over 24 inches on center.

(B) PLYWOOD. Plywood panels with a thickness of not less than 5/16 inch for 16-inch stud spacing and not less than 3/8 inch for 24-inch stud spacing.

(C) FIBERBOARD. Fiberboard panels, 4-foot by 8-foot panels, not less than 7/16-inch thick applied with the long dimension vertical on studs spaced not over 16 inches on center.

(D) GYPSUM SHEATHING. Gypsum panels not less than 1/2-inch nominal thickness on studs spaced not over 16 inches on center.

(E) PARTICLE BOARD. Particle Board Exterior Type 2-B-1 sheathing panels not less than 3/8-inch thick on studs spaced not over 16 inches on center.

(F) GYPSUM WALLBOARD. Gypsum wallboard not less than 1/2-inch thick on studs spaced not over 24 inches on center.

9.7.3 Cont.

Minimum nailing shall be as given in Tables 9-1 through 9-4. Nailing for diagonal boards shall be as specified in Sec. 9.8.3. Minimum nailing for particle board shall be the same as given for fiberboard in Table 9-3.

Sec. 9.8 ENGINEERED TIMBER CONSTRUCTION

For buildings in which a seismic analysis is required, the proportioning and design of wood systems, members, and connections shall be in accordance with the reference documents and this Section.

9.8.1 FRAMING REQUIREMENTS

All wood columns and posts shall be framed to true end bearing. Supports for columns and posts shall be designed to hold them securely in position and to provide protection against deterioration. Where post and beam or girder construction is used, positive connections shall be provided to resist uplift and lateral displacement.

9.8.2 REQUIREMENTS FOR ALL SHEAR PANELS

Horizontal and vertical shear panels shall conform to the requirements in this Section and to the requirements in the following Section pertaining to the particular type of panel.

(A) FRAMING. All framing members used in shear panel construction shall be at least 2-inch nominal in thickness.

All boundary members, chords, and collector members of shear walls and diaphragms shall be designed and detailed to transmit the induced axial forces. The boundary members shall be tied together at all corners.

Openings in diaphragms and shear walls shall be designed and detailed to transfer all shearing stresses. Where the openings would materially affect the strength of the diaphragm or shear wall, they shall be shown and fully detailed on the approved plans.

Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm or shear wall and the attached components.

(B) TORSION. Buildings that have one side without shear walls shall meet the following requirements to accomodate the indicated torsion. The diaphragm shall be sheathed with diagonal boards or plywood. The depth of the diaphragm normal to the open side shall not exceed 25 feet nor shall the ratio of depth to width exceed 1:1 for one-story-buildings or 1:1.5 for buildings over one story in height. Where calculations show that diaphragm deflections can be tolerated, the depth normal to the open side may be increased to a depth-to-width ratio not greater than 1.5:1 for conventional diagonal sheathing or 2:1 for special diagonal sheathing or plywood diaphragms. See Sec. 3.7.9.

9.8.3 DIAGONALLY SHEATHED SHEAR PANELS

Diagonally sheathed shear panels shall be constructed in accordance with the requirements of this Section for either conventional or special construction.

(A) CONVENTIONAL CONSTRUCTION. Such lumber shear panels shall be made up of 1-inch nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards shall be nailed to each intermediate bearing member with not less than two 8d nails for 1 x 6 nominal boards and three 8d nails for 1 x 8 or wider boards. One additional nail shall be provided in each board at shear panel boundaries. Where box nails are used, one additional nail shall be used at each bearing and two additional nails shall be used at shear panel boundaries. End joints in adjacent boards shall be separated by at least one stud or joist space and there shall be at least two boards between joints on the same support. Wood shear panels made up of 2-inch thick diagonal sheathing using 16d nails may be used at the same shear values and in the same locations as for 1-inch boards provided there are no splices in adjacent boards on the same support and the supports are not less than 4-inch nominal depth nor 3-inch nominal thickness.

The allowable working stress shear for conventional lumber shear panels is 200 pounds per lineal foot.

(B) SPECIAL CONSTRUCTION. Special diagonally sheathed shear panels shall conform to conventional diagonally sheathed shear panel construction and the requirements below.

Special diagonally sheathed shear panels shall be sheathed with two layers of diagonal sheathing at 90 degrees to each other on the same face of the supporting members. Each chord shall be considered as a beam loaded with uniform load per foot equal to 50 percent of the unit shear due to diaphragm action. The load shall be assumed as acting normal to the chord in the plane of the diaphragm in either direction. The span of the chord or portion thereof shall be the distance between framing members of the diaphragm such as the joists, studs, and blocking which serve to transfer the assumed load to the sheathing.

Special diagonally sheathed shear panels shall include conventional shear panels sheathed with two layers of diagonal sheathing at 90 degrees to each other on the same face of the supporting members.

The allowable working stress shear for special diagonally sheathed shear panels is 600 pounds per lineal foot.

9.8.4 PLYWOOD SHEAR PANELS

Horizontal and vertical shear panels sheathed with plywood may be used to resist shear due to earthquake forces based on the allowable working stress shear set forth in Table 9-1 for horizontal diaphragms and Table 9-2 for shear walls, or may be calculated by principles of mechanics without limitation by using values of nail strength and plywood shear values specified elsewhere in the reference standards.

(A) FRAMING. Plywood shear panels shall be constructed with plywood sheets not less than 4 feet by 8 feet, except at boundaries and changes in framing. Plywood sheets for diaphragms shall be arranged as indicated in Table 9-1. Framing members shall be provided at the edges of all sheets in shear walls. Plywood sheets shall be designed to resist shear stresses only, and chords, collector members, and boundary members shall be provided to resist axial forces resulting from the application of the seismic design forces. Boundary members shall be adequately interconnected at corner intersections.

Plywood panels less than 12-inches wide shall be blocked.

(B) NAILING. The nails and spacing of nails at shear panel boundaries and the edges of each sheet of plywood shall be as shown in Tables 9-1 and 9-2. Nails of the same size shall be placed along all intermediate framing members at 10 inches on center for floors, 12 inches for roofs, and 12 inches for walls, except that the spacing shall be 6 inches for walls of 3/8-inch plywood installed with the face grain parallel to studs which are spaced 24 inches on center.

9.8.5 SHEAR PANELS SHEATHED WITH OTHER MATERIALS

Wood stud walls sheathed with lath and plaster, gypsum sheathing board, gypsum wall board, or fiberboard sheathing may be used to resist shear due to earthquake forces in framed buildings. The allowable working stress shear values are set forth in Tables 9-3 and 9-4. Use is restricted to certain buildings and categories as contained in this Chapter. Nails shall be spaced at least 3/8 inch from the edges and ends of boards and panels. The maximum height-to-width ratio shall be 1.5:1.

The shear values tabulated shall not be cumulative with the shear value of other materials applied to the same wall. The shear values may be doubled when identical materials are applied to both sides of the wall.

9.8.6 DETAILING REQUIREMENTS

Concrete or masonry walls shall be anchored to all floors and roofs for the forces prescribed in Sec. 3.7.6. Such anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal, nor shall wood ledgers be used in cross-grain bending or tension.

TABLE 9-1

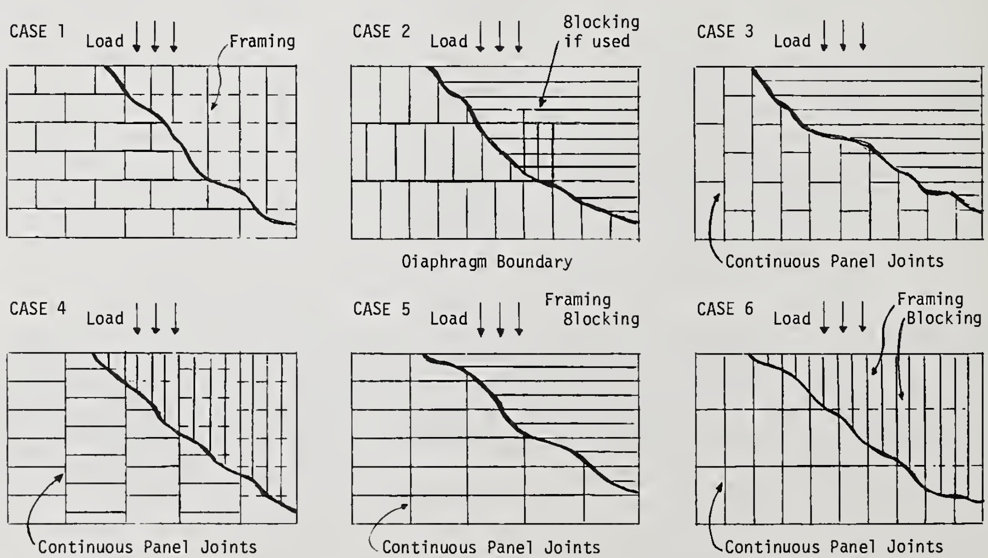
ALLOWABLE WORKING STRESS SHEAR IN POUNDS PER FOOT

FOR PLYWOOD DIAPHRAGMS[1]

PLYWOOD GRADE	Common Nail Size	Minimum Nominal Penetration in Framing (inches)	Minimum Nominal Plywood Thickness (inches)	Minimum Nominal Width of Framing Member (inches)	BLOCKED DIAPHRAGMS: Nail Spacing at Diaphragm Boundaries (all cases) and Continuous Panel Edges Parallel to Load (cases 3, 4, 5 & 6) 6 / Nail Spacing at Other Plywood Panel Edges 6	4 / 6	2-½ / 4	2 / 3	UNBLOCKED DIAPHRAGMS: Nails Spaced 6" Max. at Supported End — Load Perpendicular to Unblocked Edges and Continuous	All Other Configurations (cases 2, 3, & 4)
STRUCTURAL I	6d	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	1/2	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240
C-D, C-C, STRUCTURAL II and other similar grades	6d	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			1/2	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	1/2	2	290	385	575[2]	655[2]	255	190
				3	325	450	650	735	290	215
			3/8	2	320	425	640[2]	730[2]	285	215
				3	360	480	720	820	320	240

[1]Where boundary members provide less than 3-inch nominal nailing surface, ϕ = 0.67.

[2]Reduce tabulated allowable shears 10 percent when boundary members provide less than 3-inch nominal nailing surfaces.



NOTE: Framing may be located in either direction for blocked diaphragms.

TABLE 9-2

ALLOWABLE WORKING STRESS SHEAR FOR WIND OR SEISMIC FORCES FOR PLYWOOD SHEAR WALLS[1]

PLYWOOD GRADE	NAIL SIZE (Common or Galvanized Box)	MINIMUM NAIL PENETRATION IN FRAMING (Inches)	MINIMUM NOMINAL PLYWOOD THICKNESS (Inches)	PLYWOOD APPLIED DIRECT TO FRAMING Nail Spacing at Plywood Panel Edges (Pounds per Foot)				NAIL SIZE (Common or Galvanized (Box)	PLYWOOD APPLIED OVER ½-INCH GYPSUM SHEATHING Nail Spacing at Plywood Panel Edges (Pounds per Foot)			
				6	4	2½	2		6	4	2½	2
STRUCTURAL I	6d	1-1/4	5/16	200	300	450	510	8d	200	300	450	510
	8d	1-1/2	3/8	230[3]	360[3]	530[3]	610[3]	10d	280	430	640[2]	730[2]
	10d	1-5/8	1/2	340	510	770[2]	870[2]	-	-	-	-	-
C-D, C-D, STRUCTURAL II and Similar Grades	6d	1-1/4	5/16	180	270	400	450	8d	180	270	400	500
	8d	1-1/2	3/8	220[3]	320[3]	470[3]	530[3]	10d	260	380	570[2]	640[2]
	10d	1-5/8	1/2	310	460	690[2]	770[2]	-	-	-	-	-
	NAIL SIZE (Galvanized Casing)							NAIL SIZE (Galvanized casing)				
PLYWOOD PANEL SIDING	6d	1-1/4	5/16	140	210	320	360	8d	140	210	320	360
	8d	1-1/2	3/8	130[3]		300[3]	340	10d	160	240	360	410

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

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

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

[3]The values for 3/8-inch thick plywood applied direct to framing may be increased 20 percent provided studs are spaced a maximum of 16 inches on center or plywood is applied with face grain across studs.

TABLE 9-3

ALLOWABLE WORKING STRESS SHEARS FOR WIND OR SEISMIC LOADING ON VERTICAL SHEAR PANELS OF FIBERBOARD SHEATHING BOARD[1]

Size and Application	Nail Size	Shear Value 3-Inch Nail Spacing Around Perimeter and 6-Inch at Intermediate Points (Pounds per Foot)
7/16" x 4' x 8'	No. 11 ga. galv. roofing nail 1-1/2" long, 7/16" head	125[2]
25/32" x 4' x 8'	No. 11 ga. galv. roofing nail 1-3/4" long, 5/16" head	175

[1]Fiberboard sheathing diaphragms shall not be used to brace concrete or masonry walls.

[2]The shear value may be 175 psf for 1/2-inch x 4' x 8' fiberboard nail-base sheathing.

TABLE 9-4

ALLOWABLE WORKING STRESS SHEARS FOR SHEAR WALLS OF LATH AND PLASTER, GYPSUM SHEATHING BOARD AND GYPSUM WALLBOARD WOOD-FRAMED WALL ASSEMBLIES[1]

Type of Material	Thickness of Material	Wall Construction	Nail Spacing Maximum[2] (Inches)	Shear Value (Pounds per Foot)	Minimum Nail Size
Woven or Welded Wire Lath and Portland Cement Plaster	7/8"	Unblocked	6	180	No. 11 ga. 1-1/2" long with 7/16" diameter head nail or No. 16 ga. staples having 7/8" long legs.
Gypsum Lath, Plain or Perforated	3/8" Lath and 1/2" Plaster	Unblocked	5	100	No. 13 ga. 1-1/8" long 19/64" head, plasterboard blued nail.
Gypsum Sheathing Board	1/2" x 2' x 8'	Unblocked	4	75	No. 11 ga. 1-3/4" long, 7/16" head, diamond point, galvanized.
	1/2" x 4'	Blocked	7	175	
	1/2" x 4'	Unblocked	4	100	
Gypsum Wallboard	1/2"	Unblocked	7	100	5d cooler nails.
		Unblocked	4	125	
	1/2"	Blocked	7	125	
		Blocked	4	150	
	5/8"	Blocked	4	175	6d cooler nails
	5/8"	Blocked	Base Ply 9	250	Base Ply - 6d cooler nails.
	5/8"	Two-ply	Face Ply 7	250	Face Ply - 8d cooler nails.

[1]Shear walls shall not be used to resist loads imposed by masonry or concrete walls.
[2]Applies to nailing at all studs, top and bottom plates, and blocking.

CHAPTER 10

STEEL

Sec. 10.1 REFERENCE DOCUMENTS

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

Ref. 10.1 The American Institute of Steel Construction (AISC) Specifications (Parts 1 and 2) for the Design, Fabrication and Erection of Structural Steel for Buildings - 1969 including Supplements 1, 2 and 3.

Ref. 10.2 The Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute (AISI), 1968 Edition with Addendum No. 1.

Ref. 10.3 The Specifications for the Design of Cold-Formed Stainless Steel Structural Members, American Iron and Steel Institute (AISI), 1974 Edition.

Ref. 10.4 The Standard Specifications for Open Web Steel Joists, J- and H-Series, adopted by the Steel Joist Institute (SJI) and AISC on October 1, 1974.

Ref. 10.5 The Standard Specification for Longspan Steel Joists LJ- and LH-Series and Deep Longspan Steel Joists, DLJ- and DLH-Series, adopted by SJI and AISC on October 1, 1974.

Ref. 10.6 The Criteria for Structural Applications for Steel Cables for Buildings, AISI, 1973 Edition.

Sec. 10.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, ϕ, and the stresses permitted in the reference documents except as modified in this Section.

The value of ϕ shall be as follows:

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

10.2.1 STRUCTURAL STEEL

Reference 10.1 shall be modified as follows:

(A) LOAD COMBINATION. AISC Sec. 1.5.6 shall read as follows:

> "The strength of structural steel members for resisting seismic forces acting alone or in combination with dead and live loads shall be determined by using 1.7 times the allowable stresses in AISC Sections 1.5.1, 1.5.2, 1.5.3, and 1.5.4."

(B) SHEAR STRENGTH. In AISC Sec. 1.5.1.2 and 1.10, substitute "$F_v = 0.32\ F_y$" in lieu of "$F_v = 0.4\ F_y$" for determining shear strength.

(C) EULER STRESS. AISC Sec. 1.6.1. The definition of F'_e for the purpose of determining the strength of structural steel members shall read as follows:

"$F'_e = \dfrac{\pi^2 E}{(Kl_b/r_b)^2}$ (in the expression for F'_e, l_b is the actual length in the plane of bending and r_b is the corresponding radius of gyration. K is the effective length factor in the plane of bending.)"

(D) MEMBER STRENGTH. Amend first paragraph of AISC Sec. 2.1 by deleting "or earthquake" and adding the following:

> "The strength of members shall be determined by the requirements contained herein. Except as modified by these rules, all pertinent provisions of Part 1 shall govern."

(E) P-DELTA EFFECTS. Where axial and flexural stresses are determined considering secondary bending resulting from the design P-delta effects, all axially loaded members may be proportioned in accordance with AISC (Ref. 10.1) Sec. 1.6.1 or 2.4 except as follows:

1. The effective length factor, K, in the plane of bending may be assumed to be unity in the calculation of F_a, F'_e, P_{cr}, or P_e.

2. The coefficient C_m is computed as for braced frames.

10.2.2 COLD FORMED STEEL

References 10.2 and 10.3 shall be modified as follows:

(A) MEMBER STRENGTH. AISI Sec. 3.1.2.1 and the first paragraph of AISI Sec. 3.1.2.2 of Ref. 10.2 and AISI Sec. 3.9.1 and first paragraph of AISI Sec. 3.9.2 of Ref. 10.3 shall be modified by substituting 70 percent for the 33-1/3 percent increase to determine the strength of cold-formed members subjected to seismic forces alone or seismic forces in combination with dead and live loads.

(B) EFFECTIVE WIDTH. Modify AISI Sec. 2.3.1.1, third paragraph of Ref. 10.2 and add to AISI Sec. 2.3.1.1 of Ref. 10.3:

> "When members of assemblies are subject to stresses produced by seismic forces or seismic forces combined with dead and live loads, the effective design width, b, shall be determined using 0.60 times the stress that would be determined using the increase permitted in Sec. 3.1.2.1 or 3.1.2.2 (Sec. 3.9.1 or 3.9.2 for Ref. 10.3)."

10.2.3 STEEL CABLES

Reference 10.6 Sec. 5d shall be modified by substituting 1.5 T_4 when T_4 is the net tension in cable due to dead load, prestress, live load and seismic load. A load

10.2.3 Cont.

factor of 1.1 shall be applied to the prestress force to be added to the load combination of AISI Sec. 3.1.2 of Ref. 10.6.

Sec. 10.3 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any type of steel construction permitted in the reference documents.

Sec. 10.4 SEISMIC PERFORMANCE CATEGORY B

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

10.4.1 ORDINARY MOMENT FRAMES

Where Moment Resisting Frame Systems are used for the seismic resisting system they shall be, as a minimum, Ordinary Moment Frames designed and constructed in accordance with Part 1 of Ref. 10.1, or Ref. 10.2, or Ref. 10.3.

10.4.2 SPACE FRAMES

Space frames in building frame systems or where incorporated in bearing wall systems shall be designed and constructed in accordance with Part I of Ref. 10.1 or Ref. 10.2 or Ref. 10.3.

Sec. 10.5 SEISMIC PERFORMANCE CATEGORIES C AND D

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

10.5.1 SPECIAL MOMENT FRAMES

Where a Moment Resisting Frame System is used as the seismic resisting system, it shall be composed of Special Moment Frames conforming to the requirements of Sec. 10.6.

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

10.5.2 BRACED FRAMES

For seismic resisting systems over two stories in height using braced frames, the members shall have a compressive strength equal to at least 50 percent of the required tensile strength.

Sec. 10.6 SPECIAL MOMENT FRAME REQUIREMENTS

Special Moment Frames shall be designed in accordance with Part 2 of Ref. 10.1 with the following modifications:

1. Substitute the following for the last three paragraphs of AISC Sec. 2.1.

"Special Moment Frames shall satisfy the requirements for Type 1 construction in the plane of the frame as provided in Sec. 1.2. Type 2 construction is permitted for members between rigid frames. Connections joining a portion of a structure designed on the basis of this Part with a portion not so designed need be no more rigid than ordinary seat-and-cap angle or standard web connections. Except as modified by these rules, all other pertinent provisions of Part 1 shall govern.

The moment strength of flexural members shall be determined by the moment $M_p = ZF_y$."

2. Substitute the following for AISC Sec. 2.2:

   "Structural steel shall conform to one of the following ASTM specifications, A36-75, A441-75, A500-76, A501-76, A572-76; (Grades 42, 45, 50, and 55), or A588-75.

   EXCEPTION:
   Structural Steel ASTM A283-75 Grade D may be used for base plates."

3. AISC Sec. 2.3.1 shall not apply and the last sentence of AISC Sec. 2.3.2 shall be modified to read:

   "The axial force in the columns shall not exceed 0.6 P_y."

4. Add the following to AISC Sec. 2.4:

   "Column splices shall not be placed in an area in which a potential plastic hinge would form unless the splice fully develops the column section. Partial penetration welds shall not be used for column splices unless it can be shown that the splice strength is adequate to resist load effects of:

   a. The plastic capacity of the joints at the ends of the column with the yield strength of members assumed at 1.25 F_y, and

   b. The plastic capacity of the joint at one floor, and one-half the plastic capacity of the joint at the other floor with yield strengths of members assumed at F_y, and

   c. The load as specified in ATC Formula (3-2a)."

5. Add the following to AISC Sec. 2.5:

   "Shear in frame beams and columns and their connections shall be determined by assuming moments in members equal to the member flexural capacities at critical sections but not less than the shears resulting from elastic distribution of the specified forces.

   Beam-column joint panel zone areas shall be designed to resist the shears, f_v, based on the capacity of the members framing into the joint, but need not exceed shears produced by deforming the frame two times that resulting from the prescribed forces."

10.6 Cont.

Substitute the equation:

$$"t < \left[d_c^2 \sqrt{F_y} + \frac{180c_1 A_f}{\sqrt{1 - (fv/0.55\ F_y)^2}} \right] \frac{1}{125d_c \sqrt[4]{F_y}}"$$

in place of Equations 1.15-1 and 1.15-2 of Ref. 10.1. Applicable definitions of terms remain unchanged.

6. Add the following to AISC Sec. 2.8:

   "Beam to column connections shall develop the joint capacity determined by the strength of members framing into the joint, unless it can be shown that adequate rotation can be obtained by deformations of the connection materials.

7. Change the start of the first paragraph on Page 5-62 in AISC Sec. 2.9 to read as follows:

   "The foregoing provisions need not apply to members bending about their weak axis. However, in regions not adjacent to a plastic hinge the maximum distance...."

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 which 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 Building Code Requirements for Reinforced Concrete, American Concrete Institute. (ACI 318-71 including 1976 supplemental provisions but excluding Appendix A)

Sec. 11.2 STRENGTH OF MEMBERS AND CONNECTIONS

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

The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, ϕ, and the strengths permitted in the reference documents.

The capacity reduction factors in Ref. 11.1 Sec. 9.2 shall be modified as follows:

Connections of precast components $\phi = 0.5$

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 $0.05\ f'_c$ and special lateral reinforcement as specified in Sec. 11.7.2(C) is not provided for the full height of the component $\phi = 0.5$

For shear strength of components constructed with normal-weight aggregate concrete in buildings assigned to Seismic Performance Categories C and D where:

1. The strength of the component is governed by flexure $\phi = 0.85$
2. The strength of the component is governed by shear $\phi = 0.6$

For components constructed with light-weight aggregate concrete, the value of ϕ for shear strength shall be 80 percent of the value for normal-weight aggregate concrete. For construction joints in light-weight concrete, the value of ϕ for use in Formula 11-6 shall be 60 percent of the value for normal weight concrete.

The allowable loads on anchor bolts shall not exceed those given in Table 11-A.

Sec. 11.3 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any construction permitted in Ref. 11.1. Ordinary moment frames are frames conforming to the requirements of Ref. 11.1. Shear walls are walls conforming to the requirements of Ref. 11.1. Braced frames are frames with bracing members conforming to the requirements of Ref. 11.1.

11.3 Cont.

Anchor bolts at tops of columns and similar locations shall be closely enclosed within not less than two No. 4 or three No. 3 ties located within 4 inches from top of column.

Sec. 11.4 SEISMIC PERFORMANCE CATEGORY B

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

11.4.1 ORDINARY MOMENT FRAMES

Components of Ordinary Moment Frames constituting part of the seismic resisting system shall be modified in accordance with the requirements of Sec. 11.6.

Sec. 11.5 SEISMIC PERFORMANCE CATEGORIES C AND D

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

11.5.1 MATERIAL REQUIREMENTS

Materials used in the components of the seismic resisting system shall conform with the requirements of this Section.

The specified 28-day compressive strength, f'_c, of the concrete in special moment frames and shear walls shall not be less than 3000 psi. The specified 28-day compressive strength, f'_c, shall not exceed 4000 psi for concrete with lightweight aggregate.

Longitudinal reinforcement in special moment frames and in wall boundaries shall comply with ASTM A-706. ASTM A-615 grade 40 reinforcement may be used in these elements if (1) 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 3,000 psi) and (2) the ratio of the actual ultimate tensile stress to the actual yield stress is not less than 1.25.

11.5.2 FRAMING LIMITATIONS

Moment frames which are part of the seismic resisting system, regardless of height, shall be Special Moment Frames designed and detailed in accordance with the provisions of Sec. 11.7. Shear walls, braced frames, and diaphragms shall conform to the provisions of Sec. 11.8.

11.5.3 FRAME COMPONENTS NOT PART OF THE SEISMIC RESISTING SYSTEM

All frame components assumed not to be part of the seismic resisting system shall comply with Sec. 3.3.4(C). Such components shall satisfy the minimum reinforcement requirements specified in Chapters 7, 10, and 11 of Ref. 11.1. If nonlinear behavior is required in such components to comply with Sec. 3.3.4(C), the critical portions shall be provided with Special Lateral Reinforcement as required for Special Moment Frames in Sec. 11.7.1(B) and/or Sec. 11.7.2(C) in these provisions.

All frame components resisting axial compressive forces greater than $0.10\ f'_c A_g$ shall conform to the requirements of Sec. 11.6.2.

11.5.4 SUPPORT FOR DISCONTINUOUS COMPONENTS

Columns supporting discontinued stiff components, such as walls or trusses, where the axial design force due to seismic forces exceeds $0.05\ f'_c A_g$ shall be provided with special lateral reinforcement as specified in Sec. 11.7.2 for their full height.

Sec. 11.6 REQUIREMENTS FOR ORDINARY MOMENT FRAMES ASSIGNED TO CATEGORY B

Ordinary Moment Frames shall conform to the requirements of this Section.

11.6.1 FLEXURAL MEMBERS

Flexural members shall be provided with longitudinal reinforcement in conformance with the requirements given below.

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

The positive-moment strength at the face of the joint or at any section where flexural yielding may occur 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 component shall not be less than one-fourth the maximum moment strength provided at the face of either joint.

A flexural member framing into a column where there is no flexural member on the opposite side shall have all longitudinal reinforcement extended to the far face of of the confined region and anchored to develop the specified yield stress.

Longitudinal reinforcement reaching a column in flexural members framing into opposite sides of the column shall be continuous through the column where possible. If a top or bottom bar cannot be continued through the column because of variation in beam cross-section, it shall be extended to the far face of the confined region and anchored to develop the specified yield stress for the reinforcement.

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

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

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

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

11.6.2 MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

Members of ordinary moment frames having a design axial compressive force exceeding $0.10\ f_c'A_g$ shall have lateral reinforcement in accordance with this Section.

Lateral reinforcement in the amount specified below shall be provided over a length, l_o, 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.

Ties shall have 135-degree hooks with extensions not less than the larger of six tie diameters or 4 inches. Cross-ties may be used.

The maximum spacing shall be s_h over a length l_o measured from the joint face. The spacing s_h shall not be greater than the smaller of (a) eight diameters of the smallest longitudinal bar enclosed, (b) 24 tie diameters, or (c) 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, or (b) the maximum cross-sectional dimension of the column, or (c) 18 inches.

The first tie shall be within a distance equal to 0.5 s_h from the face of the joint. The tie spacing shall not exceed 2 s_h in any part of the member. Lateral reinforcement as required above shall be continued through the joint.

Sec. 11.7 SPECIAL MOMENT FRAME REQUIREMENTS

Special Moment Frames shall be designed and detailed in accordance with the provisions of this Section.

For determining the required lateral reinforcement in components of special moment frames in which the shear stress due to seismic forces is equal to or greater than one-half of the total design shear stress and the axial compressive force due to dead loads and seismic forces is less than 0.05 $f'_c A_g$, the concrete shear stress v_c shall be assumed to be zero.

11.7.1 FLEXURAL MEMBERS

The axial compressive force on the member shall not exceed 0.10 $f'_c A_g$. The clear span for the member shall not be less than four times its effective depth. The width-to-depth ratio shall not be less than 0.3.

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

(A) LONGITUDINAL REINFORCEMENT. Longitudinal reinforcement shall comply with the requirements of Sec. 11.6 and the additional requirements given in this Section.

Lap splicing of tensile reinforcement is permitted only if hoop or spiral reinforcement is provided over the lap length. The maximum spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 4 inches or d/4. Lap splices shall not be used (1) within the joints, (2) within a distance of twice the member depth from the face of the joint, or (3) at locations where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

Welded splices conforming to Ref. 11.1 Sec. 7.5.5.1 and approved mechanical splices conforming to Ref. 11.1 Sec. 7.5.5.2 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 component.

Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored to develop the strength of the bar by means of adequate straight embedment length and, if required, a standard hook.

(B) SPECIAL LATERAL REINFORCEMENT. Special lateral reinforcement shall comply with the requirements given below. The design shear force shall be determined assuming that ultimate resisting moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span. The ultimate resisting moments 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 $1.25f_y$.

Hoops shall be provided in the following regions:

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.

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.

Wherever compression reinforcement is required.

The first hoop shall be located not more than 2 in. from the face of a supporting member. Maximum spacing of the hoops shall not exceed (1) d/4, (2) eight times the diameter of the smallest longitudinal bars, (c) 24 times the diameter of the hoop bars, or (d) 12 inches.

Where hoops are required, longitudinal bars shall have lateral support conforming to provisions for ties in Ref. 11.1 for members subject to bending and axial load.

11.7.2 MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

Members other than walls having a design axial compressive force exceeding $0.10f'_cA_g$ are subject to the provisions of this Section.

The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 inches. The ratio of this shortest cross-sectional dimension to the orthogonal dimension shall not be less than 0.4.

(A) RELATIVE FLEXURAL STRENGTH OF COLUMNS. At any joint and in the plane of the frame considered, the moment 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 Sec. 11.7.2(C) 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.

(B) LONGITUDINAL REINFORCEMENT. Longitudinal reinforcement shall comply with the requirements given below.

The reinforcement ratio, ρ, shall not be less than 0.01 or larger than 0.06.

Lap splices are permitted only within the center half of the member span and shall be proportioned as tension splices.

Welded splices conforming to Ref. 11.1 Sec. 7.5.5.1 and approved mechanical tension-splice devices conforming to Ref. 11.1 Sec. 7.5.5.2 may be used for splicing the reinforcement provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 24 inches or longer.

(C) SPECIAL LATERAL REINFORCEMENT. Special lateral reinforcement shall have a yield strength not more than that of the longitudinal reinforcement and shall comply with the requirements of this Subsection.

Special lateral reinforcement in the amount specified below shall be provided over a length l_o 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 l_o shall not be less than the greater of (1) the depth of the member at the joint face or at the section where flexural yielding may occur, (2) one-sixth of the clear span of the member, or (3) 18 inches.

The minimum amount and arrangement of special lateral reinforcement shall conform to the following requirements:

1. The volumetric ratio of spiral or circular hoop reinforcement, ρ_s, shall be not less than that determined by the following formula:

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \tag{11-2}$$

but not less than that required by Ref. 11.1 Equation 10-3.

2. The total cross-sectional area of rectangular hoop reinforcement shall be not less than that given by Formula 11-3 or 11-4.

$$A_{sh} = 0.3\, s_h\, h_c\, \frac{f'_c}{f_{yh}} \left(\frac{A_g}{A_{ch}} - 1 \right) \tag{11-3}$$

$$A_{sh} = 0.12\, s_h h_c \frac{f'_c}{f_{yh}} \tag{11-4}$$

Special lateral reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoops may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar.

3. The maximum spacing for special lateral reinforcing shall be not more than the smaller of one-quarter of the minimum member dimension or 4 in.

4. Cross-ties or lap of overlapping hoops shall not be spaced more than 14 in. on center in the plane perpendicular to the longitudinal reinforcement.

5. Special lateral reinforcement as specified above shall be provided over the full height of all members for which the calculated point of contraflexure is not within the middle half of the member.

Special lateral reinforcement shall be provided so that the shear strength of the member will be adequate to resist a design shear determined from (1) consideration of the static forces on the member with the ultimate resisting moments acting at the faces of the joints calculated without capacity reduction factors and (2) the maximum axial compressive design force on the column.

11.7.3 JOINTS

Special lateral reinforcement as required by Sec. 11.7.2(C) shall be continued up through the joint.

The shear stress in the joint, in the direction under consideration, for joints which are laterally confined, as defined in Sec. 2.1, shall not exceed $16\sqrt{f'_c}$ for normal weight aggregate concrete nor $12\sqrt{f'_c}$ for light-weight aggregate concrete. For joints not laterally confined the allowable stresses shall be reduced 25 percent.

The shear force in the joint shall be determined from consideration of the static forces in combination with the ultimate resisting moments of the flexural members at the faces of the joints. The ultimate resisting moments of the flexural members shall be calculated without the strength reduction factor and with the assumption that the stress in the tensile reinforcement is equal to 1.25 f_y. The shear stress shall be determined on the basis of the effective section for shear, bd, for rectangular sections and A_c for other sections.

11.8 SHEAR WALLS, BRACED FRAMES, AND DIAPHRAGMS.

The requirements of this Section apply to shear walls, braced frames and diaphragms which are part of the seismic resisting system and also apply to struts, ties, and collector components which transmit axial forces induced by earthquake motions.

The minimum reinforcement ratio each way at any section of shear walls and diaphragms shall not be less than 0.0025. Reinforcement spacing each way shall not exceed 18 in. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The limiting shear stress, v_u, in shear walls and diaphragms shall be determined in accordance with the following formula:

$$v_u = 2\sqrt{f'_c} + \rho_h f_y \tag{11-5}$$

For lightweight aggregate concrete, the limiting shear stress, v_u, calculated from Formula 11-5 shall be multiplied by 0.75.

11.8.1 SHEAR WALL DETAILS AND LIMITATIONS

The reinforcement details and limitations for shear walls are given in this Section.

The reinforcement ratios ρ_n and ρ_h shall be equal. The reinforcement required by shear shall be distributed uniformly. Splices in horizontal wall reinforcement shall be staggered and splices in the two curtains, where used, shall not occur at the same location.

At least two curtains of reinforcement shall be used in a shear wall unless the unit shear is less than $2\sqrt{f'_c}$.

The average shear stress, v_u, for all wall piers sharing a common lateral force component shall not exceed $8\sqrt{f'_c}$ and v_u in any of the individual wall piers or in horizontal wall components shall be not more than $10\sqrt{f'_c}$.

Shear walls in Dual Systems and shear walls in Building Frame or Bearing Wall Systems having design compressive stresses in excess of $0.2f'_c$, as calculated for any loading combination including earthquake effects, shall have vertical boundary members along the wall edges as described in Sec. 11.8.4. The stresses shall be calculated using a linearly elastic model of the structure based on plain gross section. The vertical boundary element may be discontinued at a level where the calculated stress is less than 0.15 f'_c.

Horizontal wall reinforcement, provided to satisfy the shear or minimum reinforcement requirements, shall be anchored to the boundary elements to develop its yield strength in tension.

11.8.2 DIAPHRAGM DETAILS AND LIMITATIONS

The reinforcement details and limitations for diaphragms are given in this Section.

Diaphragms having design compressive stresses, calculated for any loading combination including earthquake effects, in excess of 0.2 f'_c shall have boundary members along the edges as described in Sec. 11.8.4. The stresses shall be calculated using a linearly elastic model of the diaphragm based on plain gross section. The boundary member may be discontinued at a point where the calculated stress is less than 0.15 f'_c.

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.

11.8.3 OPENINGS IN SHEAR WALLS AND DIAPHRAGMS

Openings in shear walls and diaphragms shall be provided with boundary members unless it is shown that the unit compressive stresses are less than 0.2 f'_c at the edges of the opening, when calculated on an elastic basis for any loading combination including earthquake effects.

11.8.4 BOUNDARY MEMBERS

Boundary members shall be provided as required by Sec. 11.8.1 and 11.8.2. Boundary members shall consist of reinforced concrete or structural-steel members encased in, or continuously attached to, the wall or diaphragm.

Structural-steel members used as boundary members shall be in conformance with Chapter 10.

Reinforced concrete boundary members shall have lateral reinforcement as specified in Sec. 11.7.2 along their full lengths.

Where vertical boundary members are required for shear walls, they shall be proportioned to resist all gravity loads on the wall and the vertical force required to resist the overturning moment related to earthquake effects.

Diaphragm boundaries shall be proportioned to resist the sum of the axial force and a force obtained as the ratio of the design moment at the section to the total depth of that section.

Boundary members around openings shall be proportioned to have a minimum total axial-force capacity in a given direction equal to the strength of the removed section. The boundary element shall be anchored to develop its yield strength in tension at the edge of the opening.

11.8.5 BRACED FRAMES

Members of braced frames, horizontal trusses, struts, ties, and and collector components shall be detailed to comply with Sec. 11.7.2. Where the members are subjected to design forces in excess of 0.2 $f_c'A_g$ including the effects of seismic forces, the detailed requirements of Sec. 11.7.2 shall apply for the full length of the member.

11.8.6 SPLICES AND ANCHORAGE

All continuous reinforcement in shear walls, diaphragms, trusses, struts, ties, chords, and collector components shall be anchored or spliced in accordance with the provisions for reinforcement in tension. The development length shall not be reduced in cases for which the provided area of steel exceeds that required.

11.8.7 CONSTRUCTION JOINTS

Construction joints in shear walls, diaphragms, and other members resisting seismic forces shall be designed and constructed to resist the design forces at the joint.

Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the total shear force across the joint shall not exceed V_j determined from the following formula:

$$V_j = \phi\ (A_{vf}f_y + 0.75\ P_n) \qquad (11\text{-}6)$$

where A_{vf} represents the total amount of reinforcement (including flexural reinforcement) normal to the construction joint acting as shear-friction reinforcement and P_n is the algebraic sum of the seismic forces and the minimum gravity loads on the joint surface acting simultaneously with the shear.

The surfaces of all construction joints in components resisting lateral forces shall be thoroughly roughened.

TABLE 11-A

ALLOWABLE SHEAR AND TENSION ON BOLTS[1]

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

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

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

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

CHAPTER 12

MASONRY

BACKGROUND

The masonry design and construction procedures given in this Chapter and Chapter 12A are essential to providing the performance levels implicit in the selection of the factors used in determining the seismic forces in these provisions. The requirements embodied in Chapters 12 and 12A have been demonstrated to be necessary by recent earthquakes and represent the latest developments in masonry construction to provide adequate seismic performance.

12.1 REFERENCE DOCUMENTS

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

12.2 STRENGTH OF MEMBERS AND CONNECTIONS

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

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

12.2.1 SPECIAL DESIGN PROCEDURES FOR UNREINFORCED MASONRY SUBJECTED TO SEISMIC FORCES.

Unreinforced masonry shall be designed in accordance with this Section.

(A) GENERAL DESIGN PROCEDURE. Unreinforced masonry designed in accordance with Sec. 12A.6.1 shall be assumed to be cracked in the tension zone. The resultant linear distribution of compressive stresses must be in equilibrium with the applied forces and the maximum compressive stress must not exceed the values of Table 12A-3.

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

12.2.1 Cont.

(B) ALTERNATE DESIGN PROCEDURE. For unreinforced masonry designed in accordance with Sec. 12A.6.2 limit the ratio of e/t for bending in one direction and the ratio R_e for bending about both principal axes to 1/6. To satisfy these ratios, the stiffness and strength of all masonry that would otherwise be in a tension zone may be ignored and the calculations carried out as if it did not exist.

> EXCEPTION:
> Cracked bed joints are not permitted in unreinforced vertical components constructed using stacked bond which are subjected to bending in the plane of the component. Therefore the procedure of the last sentence of the above paragraph is not permitted for unreinforced stacked bond components subjected to in-plane bending.

12.3 SEISMIC PERFORMANCE CATEGORY A

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

12.4 SEISMIC PERFORMANCE CATEGORY B

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

12.4.1 CONSTRUCTION LIMITATIONS

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

(A) HEIGHT LIMITATION. Components of the seismic resisting system in buildings under 35 feet in height shall be reinforced masonry when constructed using stacked bond and shall, as a minimum, be partially reinforced masonry when constructed using running bond. Components of the seismic resisting system in buildings over 35 feet in height shall be reinforced masonry and other structural components shall be partially reinforced masonry.

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

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

> EXCEPTION:
> The reinforcement provisions of Sec. 12.7.1 need not apply to partially reinforced masonry designed as unreinforced masonry.

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

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

12.4.1 Cont.

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

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

12.4.2 MATERIAL LIMITATIONS

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

Unburned clay masonry

Building Brick and Hollow Brick made from clay or shale of Grade NW

Sand-Lime Building Brick other than grades SW and MW

Concrete Building Brick and Solid Load-Bearing Concrete Masonry Units, other than Grade N

Hollow Load-Bearing Concrete Masonry Units, other than Grade N

Structural Clay Load-Bearing Wall Tile

Masonry Cement (for mortar and grout)

Mortars other than Types M or S

12.5 SEISMIC PERFORMANCE CATEGORY C

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

12.5.1 CONSTRUCTION LIMITATIONS

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

(A) REINFORCEMENT. All masonry shall be reinforced masonry.

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

(C) REINFORCED COLUMNS. In addition to the requirements of Sec. 12.6.3(F) for reinforced masonry columns, no longitudinal bar shall be farther than 6 inches from a laterally supported bar. Except at corner bars, ties providing lateral support may be in the form of cross-ties engaging bars at opposite sides of the column.

The tie spacing for the full height of masonry shear wall boundary columns and all other columns stressed by tensile or compressive axial overturning forces due to seismic effects and for the tops and bottoms of all other columns for a distance of 1/6 of clear column height but not less than 18 inches nor the maximum column dimension shall be not greater than 16 bar diameters nor 8 inches. Tie spacing for the remaining column height shall be not greater than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches.

12.5.1 Cont.

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

1. Sec. 11.8.4 when of reinforced concrete or structural steel.
2. Sec. 12.5.1(C) when of masonry.

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

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

12.5.2 MATERIAL LIMITATIONS

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

All the materials listed in Sec. 12.4.2

Structural Clay Nonload-bearing Wall Tile

Glass Units

12.6 SEISMIC PERFORMANCE CATEGORY D

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

12.6.1 CONSTRUCTION LIMITATIONS

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

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

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

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

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

12.6.1 Cont.

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

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

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

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

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

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

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

12.6.2 MATERIAL LIMITATIONS

Hollow nonload-bearing concrete masonry units shall not be used. Sand-lime building brick shall not be used for any structural masonry.

12.6.3 SPECIAL INSPECTION

Special inspection shall be provided for all structural masonry.

12.7 SHEAR WALL REQUIREMENTS

Shear walls shall comply with the requirements of this Section.

12.7.1 REINFORCEMENT

The minimum ratio of reinforcement for shear walls shall be 0.0015 in each direction. The maximum spacing of reinforcement in each direction shall be the smaller of the following dimensions: one-third the length and height of the element but not more than 32 inches. The area and spacing of reinforcement perpendicular to the shear reinforcement shall be at least equal to that of the required shear reinforcement. The portion of the reinforcement required to resist shear shall be uniformly distributed.

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

12.7.2 BOUNDARY MEMBERS

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

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

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

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

12.7.3 COMPRESSIVE STRESSES

For loading combinations including in-plane seismic forces, allowable compression stresses at any point shall not exceed those allowed for axial compression. For unreinforced masonry designed by Sec. 12A.6.1, the allowable working stress values are given in Table 12A-3. The allowable working stress values for reinforced masonry shall be the allowable working stresses given in Table 12A-5 and applicable reductions for slenderness effects shall apply. The minimum horizontal distance between lateral supports may be considered for walls as well as the minimum vertical distance. Formula 12A-7 shall not be used.

> EXCEPTION:
> For pier type wall elements that do not extend from floor to floor compression stresses under combined loading at any point may be limited to those allowed for flexural compression provided that Formula 12A-7 is also satisfied.

12.7.4 HORIZONTAL COMPONENTS

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

CHAPTER 12A

MASONRY CONSTRUCTION

Sec. 12A.1 GENERAL

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

12A.1.1 DEFINITIONS

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

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

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

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

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

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

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

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

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

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

12A.1.1 Cont.

JOINT, COLLAR. The interior longitudinal vertical joint in a wall between wythes. In grouted masonry construction, it is the grout space.

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

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

LOAD BEARING. Synonymous with Structural.

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

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

NONBEARING. This term refers to a nonload-bearing component, usually a wall.

NONLOAD-BEARING. Synonymous with Nonstructural.

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

PARTIALLY REINFORCED MASONRY. Masonry construction conforming to Sec. 12A.3.7 and other applicable provisions of this Chapter.

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

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

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

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

12A.1.2 REFERENCE DOCUMENTS

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

MATERIALS AND DESIGN	STANDARD DESIGNATION
<u>Building and Facing Brick</u>	
Clay and Shale	ASTM C62, C216, C652*
Sand-Lime	ASTM C73
Method of Test	ASTM C67
<u>Concrete Masonry Units</u>	
Hollow Load-Bearing	ASTM C90
Solid Load-Bearing	ASTM C145
Hollow Nonload-Bearing	ASTM C129
Brick	ASTM C55
Method of Test	ASTM C140
<u>Structual Clay Tile</u>	
For Walls - Load-Bearing	ASTM C34, C212
For Walls - Nonbearing	ASTM C56
For Floors	ASTM C57
<u>Cast Stone</u>	ACI 704
<u>Unburned Clay</u>	Uniform Building Code Standard 24-15
<u>Reinforcement</u>	
Reinforcing Steel	ASTM A615, A616, A617 and A706
Masonry Joint Reinforcement	ASTM A82
Welding	AWS D12.1
<u>Cement</u>	
Blended Hydraulic Cement	ASTM C595
Portland Cement and Air-Entraining Portland Cement	ASTM C150
Masonry Cement	ASTM C91
<u>Lime</u>	
Quicklime	ASTM C5
Hydrated Lime for Masonry Purposes	ASTM C207
Processed Pulverized Quicklime	ASTM C51
<u>Mortar</u>	
Other than Gypsum	ASTM C270
Aggregates for Mortar	ASTM C144
Field Tests for Mortar	Sec. 12A.8.2
<u>Grout</u>	
Aggregates for Grout	ASTM C404
Field Tests for Grout	Sec. 12A.8.2

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

Testing

Masonry Assemblies, Cores, Mortar and Grout	Sec. 12A.7 & 12A.8
Slump Test for Grout	Sec. 12A.8.2
Rate of Absorption	ASTM C67-73

12A.1.3 SYMBOLS

The symbols used in this Chapter are defined as follows:

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

A_g = Gross cross-sectional area, square inches.

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

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

C_e = Eccentricity coefficient.

C_s = Slenderness coefficient.

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

d_b = Reinforcement diameter, inches.

e = Effective eccentricity, inches.

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

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

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

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

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

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

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

f_m = Allowable compressive unit stress, psi.

f'_m = Compressive strength of masonry, psi.

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

12A.1.3 Cont.

f_t = Allowable flexural tensile stress in masonry, psi.

f_v = Allowable unit stress in web reinforcement, psi.

i = Effective length of rectangular wall element or column.

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

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

l_d = Development length, inches.

M_c = Minimum allowable moment capacity, inch-pounds.

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

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

P = Allowable vertical load, pounds.

r = Radius of gyration, inches.

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

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

t = Effective thickness, inches.

v = Shearing unit stress, psi.

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

V = Total shear, pounds.

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

12A.1.4 CRITERIA FOR MASONRY UNITS

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

The intended usage such as structural or nonstructural.

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

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

12A.1 Cont.

12A.1.5 INITIAL RATE OF ABSORPTION

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

12A.1.6 BRICK MASONRY UNIT SURFACES FOR GROUTED MASONRY

Masonry units for reinforced and unreinforced grouted masonry shall have all surfaces to which grout is to be applied capable of adhering to grout with sufficient tenacity to resist a shearing stress of 100 psi after curing 28 days. Tests, when required, shall conform to Sec. 12A.7 and 12A.8.3.

12A.1.7 RE-USE OF MASONRY UNITS

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

12A.1.8 CAST STONE

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

12A.1.9 NATURAL STONE

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

12A.1.10 GLASS BUILDING UNITS

Glass block shall have unglazed or satisfactorily treated surfaces to allow adhesion on all mortared faces. Units shall be constructed so that a minimum panel thickness of 3.5 inches can be obtained at the mortar joints.

12A.1.11 GLAZED AND PREFACED UNITS

Glazed and prefaced units shall conform to the physical criteria for unglazed and unfaced units required by this Chapter and Chapter 12 in addition to any special requirements desired for the exposed finish. Surfaces receiving mortar and surfaces to be grouted shall be unglazed.

12A.1.12 WATER

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

12A.1.13 SHRINKAGE OF CONCRETE UNITS

Concrete masonry units used for structural purposes shall have a maximum linear shrinkage of 0.065 percent from the saturated to the oven-dry condition.

12A.1 Cont.

12A.1.14 CEMENT

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

EXCEPTION:
Approved types of plasticizing agents may be added to portland cement Type I or Type II in the manufacturing process, but not in excess of 12 percent of the total volume. Plastic or waterproofed cements so manufactured shall meet the requirements for portland cement except in respect to the limitations on insoluble residue, air-entrainment, and additions subsequent to calcination.

Cements for grout shall be Type I, IA, II, IIA, III, IIIA, or V portland cement, or Type 1S, 1S-A, 1S (MS), 1S-A (MS), 1P, or 1P-A blended hydraulic cement.

12A.1.15 LIME

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

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

12A.1.16 MORTAR

Mortar shall be prepared in accordance with either of the procedures given below:

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

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

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

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

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

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

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

To maintain plasticity, mortar may be retempered with water by the method of forming a basin in the mortar and reworking it. However, any mortar which has become harsh shall not be used in the work.

12A.1.17 GROUT

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

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

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

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

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

Coarse grout may be used in grout spaces in grouted masonry 2 inches or more in width and in grout spaces in filled-cell construction having an area of 15 square inches with a least dimension of 3 inches.

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

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

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

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

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

12A.1.17 Cont.

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

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

12A.1.18 REINFORCEMENT

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

Sec. 12A.2 CONSTRUCTION

At the time of laying all masonry units shall be clean and free of dust. Burned clay and sand-lime units shall be dampened prior to laying with an absorption rate conforming to Sec. 12A.1.5. Surfaces of concrete masonry units to receive mortar shall be dampened by means of a fog spray or equivalent during hot and dry weather, as described in Sec. 12A.2.5. At the time of laying all unburned clay units shall be damp at the surface. All masonry units shall not be so wet that free water is present on the surfaces.

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

12A.2.1 JOINTS

All units shall be laid with shoved mortar joints. Solid units shall have all head and bed joints solidly filled. Except for cavity walls, spaces to be grouted, and as provided in Sec. 12A.3.3, all wall joints, collar joints, and joints between wythes shall be solidly filled.

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

Concrete abutting structural masonry such as at starter courses or at wall intersections not designed as true separation joints, shall be roughened to a full amplitude of 1/8 inch, shall be moistened per the requirements of Sec. 12A.2, and shall be bonded to the masonry per the requirements of this Chapter as if it were masonry. Unless keys are provided, vertical joints shall be considered to be stacked bond.

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

12A.2.2 BOND PATTERN

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

> EXCEPTIONS:
> 1. Rubble stone masonry joints may vary from the horizontal or vertical.
> 2. The joints in arches and similar construction may vary from the horizontal or vertical.

12A.2.2 Cont.

3. The joints in other masonry construction may vary from the horizontal or vertical provided the construction is approved in accordance with Sec. 1.5.

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

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

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

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

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

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

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

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

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

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

12A.2.3 CORBELING

Corbels in unreinforced masonry may be built only into solid masonry walls 12 inches or more in thickness. Corbels in partially reinforced masonry may be built only into masonry walls 12 inches or more in thickness unless the construction provided for the corbel is designed and constructed as reinforced masonry. The projection for each course in such corbels and in unreinforced corbels in reinforced masonry construction shall not exceed 1 inch, and the maximum projection shall not exceed 1/3 of the total thickness of the wall when used to support a chimney built into the wall. The top course of all unreinforced corbels shall be a header course.

12A.2.4 REINFORCEMENT

Reinforcement shall conform to the requirements of this Section.

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

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

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

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

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

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

12A.2.5 TEMPERATURE LIMITATIONS

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

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

12A.2 Cont.

12A.2.6 ANCHORAGE

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

12A.2.7 BOLT PLACEMENT

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

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

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

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

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

12A.2.8 PENETRATIONS AND EMBEDMENTS

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

12A.2.9 SUPPORT BY WOOD

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

Sec. 12A.3 TYPES OF CONSTRUCTION

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

12A.3.1 UNBURNED CLAY MASONRY

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

12A.3 Cont.

12A.3.2 STONE MASONRY

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

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

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

12A.3.3 SOLID MASONRY

Solid masonry shall be brick, concrete brick, or solid load-bearing concrete masonry units, laid contiguously in mortar.

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

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

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

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

12A.3.4 CAVITY WALL MASONRY

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

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

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

12A.3.4 Cont.

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

12A.3.5 GROUTED MASONRY

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

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

Grouting and construction procedures shall conform to the requirements given below.

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

1. All units in the two outer tiers shall be laid with full shoved head and bed mortar joints. Masonry headers shall not project into the grout space.

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

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

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

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

1. All units in the two tiers shall be laid with full head and bed joints.

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

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

4. The grout space (longitudinal vertical joint) shall not be less than 3 inches in width and not less than the thickness required by the placement of steel with the required clearances and shall be poured solidly with grout. Masonry walls shall cure at least three days to gain strength before grout is poured.

EXCEPTION:
If the grout space contains no horizontal steel, it may be reduced to 2 inches.

5. Vertical grout barriers or dams shall be built of solid masonry across the grout space the entire height of the wall to control the flow of the grout horizontally. Grout barriers shall be not more than 30 feet apart. Unless a true joint occurs at the barrier, reinforcement, if it is present, shall be continuous through the barrier. In work that is part of the seismic resisting system, the grout barriers shall be constructed so as to form keys, at least 3/4-inch deep, with the grout except that construction providing equivalent irregular surfaces may be used where appropriate.

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

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

8. Inspection during grouting shall be provided in accordance with Sec. 12A.7; however, the work shall not qualify for the stresses entitled, "Special Inspection", unless fully inspected per Sec. 1.6.2, 1.6.4, and 12A.7.

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

12A.3.5 Cont.

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

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

See Chapter 12 for stacked bond limitations.

12A.3.6 HOLLOW UNIT MASONRY

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

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

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

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

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

Except as provided in Chapter 12, all reinforcing except ties and masonry joint reinforcement, where permitted, shall be embedded in grout. Longitudinal horizontal reinforcing shall be placed in bond beam units, except as permitted for masonry joint reinforcement. See Sec. 12A.6.3(F) and Chapter 12.

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

12A.3.6 Cont.

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

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

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

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

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

c. Reinforcing shall be in place prior to grouting.

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

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

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

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

d. Grouting shall be done in a continuous pour, in partial lifts so as to avoid blowout of units.

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

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

12A.3.7 PARTIALLY REINFORCED MASONRY

Partially reinforced masonry is grouted masonry or hollow unit masonry containing reinforcement as specified below. Masonry joint reinforcement shall be and ties may be embedded in the mortar in the bed joints. All other reinforcement shall be embedded in grout. Minimum masonry, mortar, and grout coverages applicable to reinforced masonry shall be provided.

12A.3.7 Cont.

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

(A) REINFORCEMENT. Reinforcing for columns shall conform to the requirement of Sec. 12A.6.3(F). For walls the maximum spacing of vertical reinforcement shall be 8 feet where the nominal thickness is 8 inches or greater and 6 feet where the nominal thickness is less than 8 inches. Vertical reinforcement shall also be provided each side of each opening and at each corner of all walls. Horizontal reinforcement not less than 0.2 square inch in area shall be provided at the top of footings, at the bottom and top of wall openings, near roof and floor levels, and at the top of parapet walls and, where distributed joint reinforcement is not provided, at a maximum spacing of 12 feet where the nominal masonry thickness is 8 inches or greater and 9 feet where the nominal thickness is less than 8 inches. The vertical reinforcement ratio and the horizontal reinforcement ratio shall each be not less than 0.00027.

Where not prohibited by Chapter 12 or this Chapter, stacked bond construction may be used. When stacked bond is used the minimum horizontal reinforcement ratio shall be increased to 0.0007. This ratio shall be satisfied by masonry joint reinforcement spaced not over 16 inches or by reinforcement embedded in grout spaced not over 4 feet. Reinforcement shall be continuous at wall corners and intersections.

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

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

12A.3.8 GLASS MASONRY

Masonry of glass blocks may be used in nonloadbearing exterior or interior walls and in openings, either isolated or in continuous bands, provided the glass block panels have a minimum thickness of 3.5 inches at the mortar joint.

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

Glass block shall be laid in Types M or S mortar. Both vertical and horizontal mortar joints shall be at least 1/4 inch and not more than 3/8-inch thick and shall be completely filled.

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

Sec. 12A.4 <u>DETAILED REQUIREMENTS</u>

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

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

12A.4.1 COMBINATION OF DISSIMILAR UNITS OR CONSTRUCTION

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

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

12A.4.2 THICKNESS OF WALLS

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

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

12A.4.3 PIERS

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

12A.4.3 Cont.

Every structural pier in reinforced masonry construction whose width is between 3 and 5 times its thickness or less than 1/2 the height of adjacent openings shall have all horizontal steel in the form of ties except that in walls less than 12 inches in nominal thickness and in reinforced grouted construction such steel may be in one layer in the form of hairpins.

12A.4.4 CHASES AND RECESSES

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

12A.4.5 HOLES, PIPES, AND CONDUITS

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

12A.4.6 ARCHES AND LINTELS

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

12A.4.7 ANCHORAGE

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

12A.4.8 END SUPPORT

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

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

Anchorage to the masonry shall conform to Chapter 3.

12A.4.9 DISTRIBUTION OF CONCENTRATED LOADS

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

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

12A.4.9 Cont.

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

Sec. 12A.5 STRENGTHS AND ALLOWABLE STRESSES

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

12A.5.1 MASONRY

Except for the stresses listed in Table 12A-3 which are applicable to unreinforced masonry, the design of masonry is based on the compressive strength f'_m. The strength f'_m is reduced when the design is based on the alternate design procedure for unreinforced brick masonry of Sec. 12A.6.2. The higher stresses allowed in the Tables in this Chapter under the heading "Special Inspection Required" may only be used when all the applicable requirements for Special Inspection have been met; see Sec. 1.6 and 12A.7.

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

1. Determination of f'_m by Prism Tests. When the masonry strength is to be established by tests, the procedures shall conform to the provisions of Sec. 12A.8 with tests made both prior to and during construction.

2. Assumed Compressive Strength f'_m. When prism tests are not made as in 12A.5.1(A).1, f'_m may be assumed as listed in Table 12-4 provided other tests are made and certifications are furnished when required by the footnotes to Table 12-4 or by the provisions upon which the design is based.

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

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

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

12A.5.2 STEEL

Stresses in reinforcement shall not exceed the following:

	POUNDS PER SQUARE INCH
TENSILE STRESS:	
For deformed bars with a yield of 60,000 pounds per square inch or more and in sizes No. 11 and smaller	24,000
Joint reinforcement, 50 percent of the minimum specified yield point for the particular kind and grade of steel used, but in no case to exceed	30,000
For all other reinforcement	20,000

	POUNDS PER SQUARE INCH
COMPRESSIVE STRESS IN COLUMN VERTICALS:	
40 percent of the minimum yield strength, but not to exceed	24,000
COMPRESSIVE STRESS IN FLEXURAL MEMEBERS:	
For compressive reinforcement in flexural members, the allowable stress shall not be taken as greater than the allowable tensile stress shown above.	
The modulus of elasticity of steel reinforcement may be taken as	29,000,000 to 30,000,000

12A.5.3 BOLTS

The allowable shear loads on bolts shall not exceed the values given in Table 12A-6. See Sec. 12A.2.7 for construction requirements.

Sec. 12A.6 DESIGN REQUIREMENTS

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

12A.6.1 DESIGN PROCEDURE FOR UNREINFORCED MASONRY

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

12A.6.1(E) Cont.

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

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

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

12A.6.2 ALTERNATE DESIGN PROCEDURE FOR UNREINFORCED BRICK MASONRY

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

i is the effective length of the element.

t is the effective thickness of the element.

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

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

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

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

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

See Chapter 12 for modifications under seismic loads.

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

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

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

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

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

12A.6.3 DESIGN PROCEDURE FOR REINFORCED MASONRY

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

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

- Tensile forces are resisted only by the tensile reinforcement.
- Reinforcement is completely surrounded by and bonded to masonry material so that they will work together as a homogenous material within the range of working stresses.

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

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

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

1. Running Bond. Six times that wall thickness for construction using running bond.

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

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

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad (12A\text{-}7)$$

where:

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

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

f_b = Computed flexural unit stress.

F_b = Flexural unit stress permitted by this Chapter if member were carrying bending load only.

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

$$v = \frac{V}{bjd} \qquad (12A\text{-}8)$$

where:

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

d = The effective depth.

j = Ratio of distance between centroid of compression and centriod of tension to depth, d. $j = 0.8$ or j may be determined by a strain compatibility analysis.

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

Where the values of the shearing unit stress computed by Formula 12A-8 exceeds the shearing unit stress, v_m, masonry web reinforcement shall be provided to carry the entire stress. Web reinforcement shall cross stacked bond joints.

1. Web Reinforcement. Web reinforcement shall consist of:

a. Stirrups or web reinforcement bars perpendicular to longitudinal steel, or

b. Stirrups or web reinforcement bars archored around or beyond the longitudinal steel and making an angle of 30 degrees or more thereto, or

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

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

Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends.

2. Stirrups. The area of steel, A_v, required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by the following formula:

$$A_v = \frac{Vs}{f_v jd} \qquad (12A\text{-}9)$$

where V is the total shear, in pounds.

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

f_v is the allowable unit stress in the web reinforcement, psi.

Inclined stirrups shall be proportioned in accordance with the provisions of paragraph 3 of this Subsection.

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

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

$$A_v = \frac{V}{f_v \sin a} \qquad (12A\text{-}9a)$$

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

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

$$A_v = \frac{Vs}{f_v jd\ (\sin a + \cos a)} \qquad (12A\text{-}9b)$$

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

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

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

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

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

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

b. Stirrup area in excess of that required is provided along each terminated bar over a distance from the termination point equal to 3/4 the effective depth of the member. The stirrups shall be proportioned to provide 50 percent of the allowable shear capacity of the member based on the allowable shear stresses of Table 12A-5 for reinforcement taking no shear. The resulting spacing s shall not exceed $d/8\beta_b$ where β_d is the ratio of the area of bars cut off to the total area of bars at the section, or

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

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

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

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

$$\frac{M_c}{V} + l_a \qquad (12A\text{-}10)$$

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

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

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

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

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

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

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

f_y = the specified bar yield strength, psi.

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

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

EXCEPTIONS:

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

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

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

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

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

EXCEPTIONS:

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

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

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

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

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

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

Lengths of laps, in inches, for deformed reinforcement shall be at least $0.08\, d_b^2\, f_y/\sqrt{f_g}$ but not less than $40d_b$ for reinforcement of 40,000 psi yield strength nor less than $60\, d_b$ for reinforcement over 40,000 psi yield strength, nor less than 12 inches for reinforcing bars and 9 inches for masonry joint reinforcement. Lap lengths for plain reinforcing shall be twice that required for deformed bars but not less than 12 inches. The terms d_b, f_y, and f_g shall be as defined in Sec. 12A.6.3(D)5.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$$f_m = 0.20\, f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right] \qquad (12A\text{-}11)$$

where:

f_m = Compressive unit axial stress in masonry wall.

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

t = Thickness of wall in inches.

h = Clear distance in inches, between supporting or stiffening elements (vertical or horizontal).

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

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

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

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

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

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

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

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

The axial load on columns shall not exceed:

$$P = A_g\ (0.18\ f'_m + 0.65\ p_g\ f_s)\ [(1 - \frac{h}{40t})^3] \qquad (12A\text{-}12)$$

where:

P = Maximum concentric column axial load.

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

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

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

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

t = Least thickness of column in inches.

h = Clear height in inches.

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

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

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

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

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

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

See Chapter 12 for additional requirements, where applicable.

3. Grouting. All columns shall be grouted solid.

12A.6.4 MASONRY SHEAR WALLS

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

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

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

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

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

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

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

(B) VERTICAL TENSION AND COMPRESSION STRESSES. Except as provided for masonry designed under the alternate design procedure of Sec. 12A.6.2 as modified by Chapter 12, vertical stresses in shear walls shall be determined from the combined effects of vertical load and from the overturning effects of lateral loads. Minimum vertical loads shall be considered. Formula 3-2a shall be used for unreinforced masonry design.

12A.6.4(B) Cont.

Allowable tension stresses for unreinforced masonry shall not be exceeded. Reinforcement anchored to the foundation shall be provided to resist tension in unreinforced walls.

(C) HORIZONTAL ELEMENTS. Provisions shall be made for shear and flexural effects in horizontal elements of shear wall systems, such as beams that couple piers. For unreinforced masonry, allowable shear and tensile stresses shall not be exceeded. Tensile reinforcing and shear reinforcing, if required, shall be provided for reinforced masonry. In reinforced masonry, when the horizontal span of the element is less than twice the total height of the element, shear reinforcing shall be in the form of diagonal bars extending from corner to corner with complete anchorage to the pier elements or shall be web reinforcing conforming to Sec. 12A.6.3(C).

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

Shear resistance of masonry shall be based on net areas parallel to the shear. Both vertical and horizontal shear shall be considered including the net bedded area, the net cross-sectional area of hollow units, and the net vertical shear area. Where only partial mortar coverage is provided, such as in hollow unit construction where only the face shells in the bed joints and partial head joint coverage is usually specified, only the actual specified mortar coverage shall be considered effective. However, continuous vertical and horizontal grout elements may be considered as part of the net areas.

For reinforced masonry, the shear stress shall be computed by Formula 12A-8. Horizontal shear reinforcing, when required, shall be provided with that portion required to resist shear uniformly distributed and spaced out not more than 1/3 the wall depth or as required by Sec. 12.7, whichever is less.

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

12A.6.5 SCREEN WALLS

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

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

The maximum size of panels shall be 144 square feet with the maximum dimension in either direction of 15 feet. Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel. Supports at the top and ends of the panel shall be by means of confinement of the masonry by at least 1/2 inch into and between the flanges of a steel channel. The space between the end of the panel and the web of the channel shall be at least 1/2 inch and shall be void of mortar. The use of equivalent configuration in other steel sections or in masonry or concrete is acceptable.

Horizontal and vertical joints shall be not less than 1/4 inch thick. All joints shall be completely filled with mortar and shall be shoved joints.

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

12A.6.5 Cont.

(B) REINFORCED PANELS. All panels in Categories B, C, or D construction shall be reinforced per Sec. 12.4.1(D).

Sec. 12A.7 SPECIFIC INSPECTIONS, SPECIAL INSPECTIONS, AND TESTS

Specific and Special Inspections shall be provided and Tests shall be made in accordance with the requirements of this Section. The Regulatory Agency may for masonry work which it determines to be minor in nature waive requirements for certifications, Specific Inspections, Tests, Special Inspection, or some items of Special Inspection. The Special Inspections and Tests of Sec. 12A.7.2, where applicable, shall be provided for all parts of masonry construction. The Special Inspection requirements of Sec. 1.6.2 are in addition to Sec. 12A.7.2 and apply only to the designated seismic system.

Specific and Special Inspection shall be done to an extent that the Inspector(s) or testing agency can certify to the requirements of Sec. 1.6.4. In general, for large jobs or for moderate size jobs, this will require continuous observation during the masonry work. However, some inspections may be done on a periodic basis provided they satisfy the requirements of this Chapter and provided this periodic scheduled inspection is performed as outlined in the project design documents or the approved Quality Assurance Plan.

12A.7.1 SPECIFIC INSPECTIONS AND TESTS

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

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

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

12A.7.2 SPECIAL INSPECTION AND TESTS

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

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

- For the examination of materials and/or certifications of materials for compliance.
- For the observation of measurement and mixing of field-mixed mortar and grout including checks on consistency.
- For the determination of the moisture conditions of the masonry units at the time of laying.
- For periodic observation of the laying of masonry units with special attention to joints including preparations prior to buttering, portions to be filled, shoving, etc.

- For observation of the bonding of units in the walls between wythes and at corners and intersections.

- For the proper placement of reinforcement including splices, clearances, and support.

- For observation of the construction of chases, recesses, and the placement of pipes, conduits, and other weakening elements.

- For inspection of grout spaces immediately prior to grouting including the removal of mortar fins as required, removal of dirt and debris, and the conditions at the bottom of the grout space. For high lift work this shall be done prior to the closing of cleanouts and shall also include the proper sealing of cleanouts.

- For the preparation, or supervision of preparation, of required samples such as mortar, grout and prisms.

- For the observation of grout placement with special attention to procedures to obtain filling of required spaces, the avoidance of segregation, and proper consolidation and reconsolidation.

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

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

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

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

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

- For unreinforced or reinforced grouted masonry, one series of core tests for shear bond shall be made for each 5000 square feet of wall or equivalent.

12A.7.2 Cont.

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

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

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

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

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

12A.7.3 LOAD TESTS

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

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

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

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

L = span of the member in feet

t = thickness or depth of the member in feet.

12A.7.4 REPORTING

Reporting and compliance procedures shall conform to Sec. 1.6.4.

Sec. 12A.8 TEST CRITERIA

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

12A.8.1 MASONRY PRISMS

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

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

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

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

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

where:

H = height of specimen in inches
d = minimum dimension of specimen in inches
Intermediate values may be interpolated.

(A) STORAGE OF TEST PRISMS. For storage of test prisms follow Method B of ASTM E477 except as modified herein. Test prisms made in the laboratory shall be stored for seven days in air, at a temperature of 70 degrees plus or minus 5 degrees, in a relative humidity exceeding 90 percent; and then in air at a temperature of 70 degrees plus or minus 5 degrees, at a relative humidity of 30 percent to 50 percent until tested.

Test prisms made in the field shall be stored undisturbed for 48 to 96 hours in the field under the same conditions, insofar as possible, and adjacent to the work they are to represent. They may be covered with wood or damp burlap, but such covering shall not shade the sides from the sun. After field storage, they shall be transported to the laboratory for continued curing as specified for laboratory constructed prisms. Field curing may continue as specified for the initial seven days.

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

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

Prisms shall be capped and tested in compression. The standard age of test specimens shall be 28 days, but 7-day tests may be used provided the relation between the 7-day and 28-day strengths of the masonry is established by adequate test data for the materials used.

12A.8.1 Cont.

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

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

12A.8.2 TESTS FOR GROUT AND MORTAR

Tests for grout and mortar shall conform to this Section.

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

(B) MORTAR SAMPLES FOR COMPRESSION TESTS. Spread mortar on the masonry units 1/2-inch to 5/8-inch thick. Place a masonry unit on top of the mortar and allow to stand for two minutes. Immediately remove mortar and place in a 2-inch by 4-inch cylinder in two layers, compressing the mortar into a cylinder using a flat-end stick or fingers. Lightly tap mold on opposite sides, level off, and immediately cover molds and keep them damp until taken to the laboratory. After 48-hours set, have the laboratory remove molds and place them in the fog room until tested in the damp condition.

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

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

12A.8.3 CORE TESTS FOR SHEAR BOND

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

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

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

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

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

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

The unit shear strength shall be calculated and reported as the maximum load divided by the shear area. Visual examination of all cores shall be made to ascertain if the joints are filled. The report shall include the results of these examinations and the condition of all cores cut on each project regardless of whether or not the core specimens failed during the cutting operation.

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

TABLE 12A-1A

COMPRESSIVE STRENGTH OF MORTAR CLASSIFIED
IN ACCORDANCE WITH PROPERTY SPECIFICATIONS
(Pounds per Square Inch)

Mortar Type	Average Compressive Strength at 28 Days
M	2,500
S	1,800
N	1,500

TABLE 12A-1B

MORTAR PROPORTIONS FOR MORTAR CLASSIFIED
IN ACCORDANCE WITH PROPORTIONS CLASSIFICATIONS
(Parts by Volume)

Mortar Type	Portland Cement	Masonry Cement	Hydrated Lime or Lime Putty[1]	Aggregate Measured in a Damp, Loose Condition
M	1	1	--	Not less than 2½ and not more than 3 times the sum of the volumes of the cements and lime used.
	1	--	¼	
S	½	1	--	
	1	--	over ¼ to ½	
N	--	1	--	
	1	--	over ½ to 1¼	

[1]When plastic or waterproof cement is used as specified in Sec. 12A.1.14, hydrated lime or putty may be added but not in excess of one-tenth the volume of cement.

TABLE 12A-2

MINIMUM THICKNESS OF MASONRY WALLS

TYPE OF MASONRY	Maximum thickness ratio unsupported height or length to Thickness[1]	NOMINAL MINIMUM THICKNESS (INCHES)[3] Walls whose only structural function is exterior enclosure, nonstructural walls, and partitions	Other Structural Uses: Thickness for the uppermost 35[6] foot high portion of wall	Other Structural Uses: Thickness increase for each 35 feet or fraction thereof below the uppermost 35[6] foot high portion of wall
STRUCTURAL WALLS:				
Unburned Clay Masonry	10	16	16	--
Stone Masonry	14	16	16	4
Cavity Wall Masonry	20[5]	8	12[7]	4
Hollow Unit Masonry	20	8	12[7]	4
Solid Masonry	20	8	12[7,8]	4
Grouted Masonry	20[2]	6	10[7,8]	4
Reinforced Grouted Masonry	25[2,9]	6	6	--
Reinforced Hollow Unit Masonry	25[2,9]	6	6	--
NONSTRUCTURAL AND PARTITIONS:[4]				
Unreinforced	36[5]	2	--	--
Reinforced	48	4	--	--

[1]For cantilever walls, the actual height or length, as applicable, used to compute the actual thickness ratio shall be doubled.
[2]If the only structural function of the wall is the enclosure of a building's exterior, the maximum thickness ratio may be increased to 22 for grouted masonry and 36 for reinforced walls.
[3]The minimum thickness requirements of Sec. 12A.3 shall also be satisfied.
[4]The thickness of plaster coatings may be considered in satisfying thickness ratios and minimum thickness requirements but shall not be used to take stresses.
[5]In determining the thickness ratio for cavity walls, an effective thickness shall be used.

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

$$T = 2/3\ (T_o - w_c)$$

where T_o = overall thickness of wall, including width of cavity,

w_c = width of cavity

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

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

[6]Seventy feet for stone masonry.
[7]These thicknesses may be reduced to 8 inches for walls that are not over 35 feet in total height in buildings that are not over three stories high.
[8]These thicknesses may be reduced to 6 inches for grouted walls and 8 inches for solid masonry walls in one-story buildings when the wall is not over 9 feet in total height, provided that when gable construction is used an additional 6 feet in height is permitted to the peak of the gable.
[9]Except for partially reinforced masonry, the maximum thickness ratios for one-story walls designed as deep beams may be increased to 36.

TABLE 12A-3

ALLOWABLE WORKING STRESSES IN UNREINFORCED MASONRY

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

[1]Allowable axial or flexural compressive stresses in psi gross on cross-sectional area (except as noted). The allowable working stresses in bearing directly under concentrated loads may be 50 percent greater than these values. Allowable axial stresses are only applicable if the maximum thickness ratios of Table 12A-2 are not exceeded. Reduce these values by 20 percent when designing columns.

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

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

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

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

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

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

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

TABLE 12A-4

ASSUMED COMPRESSIVE STRENGTH OF MASONRY

f'_m - psi

TYPE OF UNIT	COMPRESSIVE STRENGTH OF UNITS, psi OR GRADE	f'_m[3]		
		TYPE N MORTAR	TYPE S MORTAR	TYPE M MORTAR
Solid Clay[4]	14,000 psi gross[1]	4300[2,5]	5300[2,5]	6300[2,5]
	12,000 psi gross[1]	3800[2,5]	4600[2,5]	5500[2,5]
	10,000 psi gross[1]	3300[2,5]	4000[2,5]	4600[2,5]
	8,000 psi gross[1]	2700[2,5]	3300[2,5]	3800[2,5]
	6,000 psi gross[1]	2200[5]	2600[5]	3000[2,5]
	4,000 psi gross[1]	1600	1900	2200[5]
	2,000 psi gross[1]	1100	1200	1300
Solid Units other than Clay	6,000 psi gross[1]	1350	1900	2400
	4,000 psi gross[1]	1250	1650	2000
	2,500 psi gross[1]	1100	1350	1550
	1,500 psi gross	875	1025	1150
Hollow Concrete	Gd. N	--	1350	1350
Hollow Concrete - Grouted Solid	Gd. N	--	1500	1500
Hollow Clay	Gd. LB with 1-¼" Min face Shell	--	1350	1350
Hollow Clay - Grouted Solid	Gd. LB with 1-¼" Min face Shell	--	1500	1500
Hollow Clay Brick	5,000 psi net[1]	--	2500[5]	2500[5]
Hollow Clay Brick - Grouted or Reinforced	Type I	--	2000	2000

[1]When the required strength of the units exceeds 3000 psi, compression tests of the units conforming to the applicable reference documents and Sec. 12A.7 shall be made. These tests shall not be required if certifications conforming to Sec. 12A.7 and Sec. 12A.8 and acceptable to the Regulatory Agency are provided during construction.
[2]When the assumed f'_m exceeds 2600 psi, prism tests conforming to Sec. 12A.7 and Sec. 12A.8 shall be provided during construction. Certification of the units is not acceptable in lieu of tests.
[3]Intermediate values may be interpolated.
[4]When the alternate design procedure for unreinforced brick masonry of Sec. 12A.6.2 is used for design the units shall comply with the dimension and distortion tolerances specified for type FBS. Where such brick do not comply with these requirements, the compressive strength of brick masonry shall be determined by prism tests as required by Sec. 12A.5.1(A)1.
[5]Where grouted construction is used, the value of f'_m shall not exceed the compressive strength of the grout unless prism tests conforming to Sec. 12A.7 and 12A.8 are provided during construction. As an alternative, the grout strength may be specified at not less than the value of f'_m with grout tests conforming to Sec. 12A.7 and 12A.8 provided during construction for verification.

TABLE 12A-5

ALLOWABLE WORKING STRESSES (PSI) FOR REINFORCED MASONRY

TYPE OF STRESS	REINFORCED GROUTED AND HOLLOW UNIT MASONRY SPECIAL INSPECTION REQUIRED	
	YES	NO
Compression-Axial, Walls	See Section 12A.6.3(E)	One-half of the values permitted under Section 12A.6.3(E)
Compression-Axial, Columns	See Section 12A.6.3(F)	One-half of the values permitted under Section 12A.6.3(E)
Compression-Flexural	$0.33\ f'_m$ but not to exceed 900	$0.166\ f'_m$ but not to exceed 450
Shear:		
Reinforcement taking no shear[3]		
Flexural[2]	$1.1\ \sqrt{f'_m}$ 50 Max.	25
Shear walls[3,4] $M/VD \geq 1$[6]	$.9\ \sqrt{f'_m}$ 40 Max.	20
$M/Vd = 0$[6]	$2.0\ \sqrt{f'_m}$ 50 Max.	25
Reinforcing taking all shear		
Flexural	$3.0\ \sqrt{f'_m}$ 150 Max.	75
Shear walls[4] $M/Vd \geq 1$[6]	$1.5\ \sqrt{f'_m}$ 75 Max.	35
$M/Vd = 0$[6]	$2.0\ \sqrt{f'_m}$ 120 Max.	60
Modulus of Elasticity	$600\ f'_m$ but not to exceed 3,000,000	$500\ f'_m$ but not to exceed 1,500,000
Modulus of Rigidity	$240\ f'_m$ but not to exceed 1,200,000	$200\ f'_m$ but not to exceed 600,000
Bearing on full Area[5]	$0.25\ f'_m$ but not to exceed 900	$0.125\ f'_m$ but not to exceed 450
Bearing on 1/3 or less of area[5]	$0.30\ f'_m$ but not to exceed 1200	$0.15\ f'_m$ but not to exceed 600

[1]Stresses for hollow unit masonry are based on net section.
[2]Web reinforcement shall be provided to carry the entire shear in excess of 20 pounds psi whenever there is required negative reinforcement for a distance of 1/16 the clear span beyond the point of inflection.
[3]Allowable shear resisted by the masonry where lightweight concrete units are used is limited to 85 percent of the tabulated values.
[4]Interpolate by straight line for M/Vd values between 0 and 1.
[5]This increase shall be permitted only where the least distance between the edges of the loaded and unloaded areas is a minimum of ¼ of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonable concentric area greater than 1/3, but less than the full area, shall be interpolated between the values given.
[6]M is the maximum bending moment occurring simultaneously with the shear load V at the section under consideration.

TABLE 12A-6

ALLOWABLE SHEAR ON BOLTS[1]

	UNBURNED CLAY UNITS		ALL OTHER MASONRY		
				SHEAR	
DIAMETER OF BOLTS (Inches)	MINIMUM EMBEDMENT (Inches)	SHEAR (Pounds)[2]	MINIMUM EMBEDMENT (Inches)	SOLID MASONRY (Pounds)[2]	GROUTED CONSTRUCTION (Pounds)[2]
1/4	--	--	4	--	180
3/8	--	--	4	--	270
1/2	--	--	4	230	370
5/8	12	130	4	330	500
3/4	15	200	5	500	730
7/8	18	270	6	670	1000
1	21	330	7	830	1230[3]
1-1/8	24	400	8	1000	1500[3]

[1]Edge distance shall be not less than 2 inches nor 5 bolt diameters for edges parallel to the direction of stress. Edge distances shall be not less than 3 inches nor 6 bolt diameters for edges perpendicular to the direction of stress. Center to center spacings shall be not less than 12 bolt diameters.
[2]The tabulated values are for construction where Special Inspection is not provided. Where Special Inspection is provided 150 percent of these values are permitted.
[3]These values are permitted only with units having a minimum compressive strength of 2500 pounds per square inch or more.

TABLE 12A-7

ALLOWABLE STRESSES TO BE USED WITH THE ALTERNATE DESIGN PROCEDURE FOR UNREINFORCED BRICK MASONRY[1]

DESCRIPTION		ALLOWABLE STRESSES, PSI WITHOUT SPECIAL INSPECTION	ALLOWABLE STRESSES, PSI WITH SPECIAL INSPECTION
Compressive, Axial[2]			
Walls	f_m	$0.13\ f'_{mb}$	$0.20\ f'_{mb}$
Columns	f_m	$0.10\ f'_{mb}$	$0.16\ f'_{mb}$
Compressive, Flexural[2]			
Walls	f_m	$0.21\ f'_{mb}$	$0.32\ f'_{mb}$
Columns	f_m	$0.17\ f'_{mb}$	$0.26\ f'_{mb}$
Tensile, Flexural[5,6]			
Normal to bed joints[2]			
M or S mortar	f_t	18	36
N mortar	f_t	14	28
Parallel to bed joints[3]			
M or S mortor	f_t	36	72
N mortar	f_t	28	56
Shear[7]			
M or S mortar	V_m	$0.3\sqrt{f'_{mb}}$ but not exceed 35	$0.3\sqrt{f'_{mb}}$ but not to exceed 75
N mortar	V_m	$0.3\sqrt{f'_{mb}}$ but not to exceed 28	$0.3\sqrt{f'_{mb}}$ but not to exceed 56
Bearing			
On full area	f_m	$0.17\ f'_{mb}$	$0.25\ f'_{mb}$
On one-third area or less[4]	f_m	$0.21\ f'_{mb}$	$0.375\ f'_{mb}$
Modulus of Elasticity	E_m	$500\ f'_{mb}$ but not to exceed 1,500,000 psi	$600\ f'_{mb}$ but not to exceed 3,000,000 psi
Modulus of Rigidity	E_r	$200\ f'_{mb}$ but not to exceed 600,000 psi	$240\ f'_{mb}$ but not to exceed 1,200,000 psi

[1]See Section 12A.6.2.
[2]Direction of stress is normal to bed joints; vertically in normal construction.
[3]Direction of stress is parallel to bed joints; horizontally in normal masonry construction. If masonry is laid in stacked bond, tensile stresses in the horizontal direction shall not be permitted in the masonry.
[4]This increase shall be permitted only when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area shall be interpolated between the values given.
[5]For computing the flexural resistance of cavity walls, the lateral load shall be distributed to the wythes according to their respective flexural rigidities.
[6]In the use of these allowable stresses, consideration shall be given to the influence of unusual vibration and impact forces.
[7]See Section 12A.6.3(c).

INTRODUCTION TO CHAPTER 13

SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

Chapter 13 is included in these provisions for the use of those jurisdictions that are interested in a program of systematic abatement of seismic hazards in existing buildings. The following amendments to Chapters 1 and 2 are required if Chapter 13 is adopted.

AMENDMENTS TO CHAPTER 1

Add to Sec. 1.2 SCOPE. To the second paragraph:

"These provisions establish criteria for the evaluation of potential hazards in existing buildings, and where required the level of strengthening of these buildings or abatement of potential hazards therein."

Modify Sec. 1.3.4 to read as follows:

"1.3.4 SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS.

Seismic hazards in existing buildings shall be identified and the hazards abated in accordance with the requirements of Chapter 13."

AMENDMENTS TO CHAPTER 2

Add to Sec. 2.1 DEFINITIONS.

"OCCUPANCY POTENTIAL, OP, is a measure of the number of persons that might be in a building at any given time.

HISTORICAL BUILDING is a building that has been designated by a local, state, or federal jurisdiction as having special historical significance."

Add to Sec. 2.2 SYMBOLS.

"r_c = The earthquake capacity ratio as determined in Sec. 13.2.2.

t_x = The length of time in years permitted for the abatement of potential seismic hazards in buildings as defined in Sec. 13.3.2.

V_{as} = The seismic shear force capacity computed for the existing system or component, as defined in Sec. 13.2.2.

V_{rs} = The seismic shear force that the existing system or component would be required to resist in order to meet the requirements of these provisions for a new building; see Sec. 13.2.2.

α_t = The coefficient, expressed in years, specified by the cognizant jurisdiction for determining t_x; see Sec. 13.3.2."

CHAPTER 13: SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

Sec. 13.1 GENERAL

This Chapter includes procedures for the evaluation of seismic hazards in existing buildings and the determination of the extent of remedial work required.

The Cognizant Jurisdiction shall:

Identify types of buildings which require evaluation.

Perform Qualitative Evaluations, or require the owners to submit reports of qualitative evaluations prepared by registered engineers or architects, of buildings assigned to Seismic Performance Category C and identified as requiring Evaluation.

Require Analytical Evaluations prepared by registered structural engineers for all buildings assigned to Seismic Performance Category D and for those buildings assigned to Seismic Performance Category C whose degree of hazard is judged uncertain by Qualitative Evaluation.

Require that, where potential seismic hazards in buildings have been determined to exist, the hazards be abated by removal or by strengthening in accordance with Sec. 13.3.

13.1.1 IDENTIFICATION OF BUILDINGS REQUIRING EVALUATION

Evaluation shall be required for all buildings designed before 19__* and located in areas having a Seismicity Index of 4.

Evaluation shall also be required for buildings designed after 19__* and located in areas having a Seismicity Index of 4 which incorporate features and/or materials proved vulnerable in earthquakes or which have had their effective seismic resisting systems significantly weakened since construction.

The provisions contained in this Chapter do not apply to the evaluation of damaged buildings under emergency conditions immediately following an earthquake.

Buildings assigned to Seismic Performance Category C and having an Occupancy Potential of 100 or less are not subject to these provisions except that exterior nonstructural elements must be evaluated for all buildings in Category C. The Occupancy Potential, OP, shall be determined in accordance with the following formula:

$$OP = \frac{\text{Total area of all floors}}{SFPO} \qquad (13\text{-}1)$$

where:

SFPO = The square feet per occupant for each floor as given in Table 13-A or as established by the Cognizant Jurisdiction

OP is the potential number of persons in a building used as an index of life hazard

*Dates to be determined by the Regulatory Agency (see Commentary).

Sec. 13.2 EVALUATION OF SEISMIC HAZARDS IN EXISTING BUILDINGS

Procedures for evaluation of seismic hazards are given in this Section.

13.2.1 QUALITATIVE EVALUATION

Qualitative Evaluation shall be carried out for the primary structural system and for each exterior or interior nonstructural system or component which may pose a potential seismic hazard, based on examination of available pertinent design, construction, and installation documentation and on-site inspection.

Qualitative Evaluation shall determine which of the following is appropriate:

1. The design and detailing are, in the judgement of the professional making the evaluation, capable of meeting the requirements of these provisions or

2. The seismic resistance capacity can not be determined on the basis of the information available and an Analytical Evaluation is required.

Exterior nonstructural components which should be evaluated for potential seismic hazard include, but are not limited to, parapets, ornamentation, marquees, overhangs, or other appendages or facings which are located on the outside of the building.

Interior nonstructural components which should be evaluated for potential seismic hazard include, but are not limited to, equipment, lights, ornamentation, and finishes which, because of their weight, could cause injury if they were dislodged during an earthquake. Interior nonstructural components whose failure in an earthquake could block exits or stairs or result in fire or the release of toxic materials shall also be evaluated. Interior nonstructural components in buildings assigned to Seismic Performance Category C need not be evaluated if the primary structural systems in the buildings have been found to be in conformance with these provisions.

The report of Qualitative Evaluation shall include sketches showing the dimensions of the basic structural systems that transfer earthquake forces. The sketches shall also include cross-sectional details for critical members, typical joint details, and any other details considered essential for the structural system to perform satisfactorily during earthquakes. If a structural system or nonstructural system or component is determined to be capable of meeting the requirements of these provisions, the specific reasons for the determination shall be provided in the report.

13.2.2 ANALYTICAL EVALUATION

Analytical evaluations shall determine earthquake resistance of primary structural systems and nonstructural components. The analysis shall be based insofar as possible on the same general procedures and assumptions recommended in the preceding Chapters for the seismic design of new buildings. When these design provisions for new buildings are not applicable to existing buildings, deviations may be permitted by the Regulatory Agency. All such deviations shall be justified in the report of Analytical Evaluation.

Seismic resistance shall be expressed in terms of earthquake capacity ratio, r_c. This earthquake capacity ratio is the ratio of the capacity of the existing building to resist the effects of earthquake motions to the required capacity for new buildings as given in these provisions. The earthquake effects are related to the earthquake base shear, and the value of the capacity ratio, r_c, shall be determined by the following formula:

13.2.2 Cont.

$$r_c = V_{as}/V_{rs} \qquad (13\text{-}2)$$

where:

V_{as} = The seismic shear force capacity computed for the existing building, system, or component

V_{rs} = The seismic shear force that the existing system or component would be required to resist in order to meet the requirements of these provisions for a new building. For the building as a whole this shear force is the base shear V or $\overline{V}$ as determined in Sec. 4.2 or 5.8, respectively.

The seismic shear capacity, V_{as}, may be governed by the available resistance to shear, moment, or axial forces or by drift limitations.

Primary structural systems and nonstructural components when subjected to Analytical Evaluation are defined as not conforming to the recommendations of this Chapter if the earthquake capacity ratio, r_c, is less than a minimum value. The minimum acceptable earthquake capacity ratios for various systems and components are as follows:

For primary structural systems and nonstructural components of buildings assigned to Seismic Performance Category D	$r_c = 0.5$
For primary structural systems and interior nonstructural components of buildings assigned to Seismic Performance Category C, the smaller of the following values	$r_c = 0.5$ or
	$r_c = 0.25(1 + \frac{OP-100}{700})$
For exterior nonstructural components of buildings assigned to Seismic Performance Category C	$r_c = 0.5$

The report of analytical evaluation shall include diagrams showing complete details of the basic structural systems that transfer earthquake forces. The report shall also include the calculations for determining the earthquake capacity ratios stating any assumptions that are not normally made in the design of new buildings. For those structural systems and nonstructural components not conforming to the recommendations of this Chapter, the time permitted to complete remedial measures computed in accordance with Sec. 13.3.2 shall be reported.

Sec. 13.3 HAZARD ABATEMENT MEASURES

The level of forces which existing buildings and components shall be capable of resisting and the time limits for completing the required hazard abatement measures are to be determined in accordance with this Section.

13.3.1 GENERAL

When a primary structural system or a nonstructural system or component does not conform to the requirements of this Chapter, measures shall be taken to abate the nonconformance. Primary structural systems and nonstructural components classified as potential seismic hazards as a result of the evaluation made in accordance with Sec. 13.2 may have the hazard abated by strengthening so that the applicable earthquake capacity ratio is increased to the following level:

13.3.1 Cont.

For primary structural systems and nonstructural components of buildings assigned to Seismic Performance Category D	$r_c = 1.0$
For primary structural systems and interior nonstructural components of buildings assigned to Seismic Performance Category C, the smaller of the following values	$r_c = 1.0$ or
	$r_c = 0.5(1 + \frac{OP-100}{700})$
For exterior nonstructural components of buildings assigned to Seismic Performance Category C	$r_c = 1.0$

The potential seismic hazard may also be abated by removing components classified as hazardous or by demolition where primary structural systems have been classified as hazardous.

EXCEPTION:

The requirements may be modified for designated Historical Buildings.

In the case of Historical Buildings alternate materials, methods of construction, and analyses may be used as permitted by Sec. 1.5 and 3.5.

13.3.2 PERMISSIBLE TIMES TO COMPLETE SEISMIC HAZARD ABATEMENT MEASURES.

The length of time in years, t_x, permitted for the abatement of potential seismic hazards in buildings shall be determined as follows:

1. For buildings assigned to Seismic Performance Category D and having one or more nonconforming primary structural systems, t_x shall not exceed the larger of the following	$t_x = 1.0$ or
	$t_x = \alpha_t r_c < 15$
2. For buildings assigned to Seismic Performance Category D and having adequate primary structural systems but having nonconforming interior nonstructural components	$t_x = 2.0$
3. For primary structural systems and interior nonstructural components in buildings assigned to Seismic Category C, t_x shall not exceed the larger of the following	$t_x = 2.0$ or
	$t_x = \alpha_t\ (1 + \frac{200}{OP}) r_c < 15$
4. For exterior nonstructural components on all buildings	$t_x = 1.0$

For the purpose of evaluating the permissible time in 1 and 3 above, r_c is the least value of the earthquake capacity ratio computed for a primary structural system. For buildings classified as nonconforming by Qualitative Evaluation r_c shall be 0.1. The value of α is ___* years.

*To be determined by the Regulatory Agency (see Commentary).

TABLE 13-A

SUGGESTED SQUARE FEET PER OCCUPANT PER FLOOR

USE	SQUARE FEET PER OCCUPANT
1. Aircraft Hangars (no repair)	500
2. Auction Rooms	7
3. Assembly Areas, Concentrated Use (without fixed seats)	7
Auditoriums	
Bowling Alleys (assembly areas)	
Churches and Chapels	
Dance Floors	
Lodge Rooms	
Reviewing Stands	
Stadiums	
4. Assembly Areas, Less-concentrated Use	15
Conference Rooms	
Dining Rooms	
Drinking Establishments	
Exhibit Rooms	
Gymnasiums	
Lounges	
Skating Rinks	
Stages	
5. Children's Homes and Homes for the Aged	80
6. Classrooms	20
7. Dormitories	50
8. Dwellings	300
9. Garage, Parking	200
10. Hospitals and Sanitariums-Nursing Homes	80
11. Hotels and Apartments	200
12. Kitchen - Commercial	200
13. Library Reading Room	50
14. Locker Rooms	50
15. Mechanical Equipment Room	300
16. Nurseries for Children (day care)	50
17. Offices	100
18. School Shops and Vocational Rooms	50
19. Stores - Retail Sales Rooms	
Basement	20
Ground Floor	30
Upper Floors	50
20. Warehouses	300
21. All Others	100

CHAPTER 14

GUIDELINES FOR REPAIR AND STRENGTHENING OF EXISTING BUILDINGS

Sec. 14.1 GENERAL

This Chapter contains guidelines and commentary for repair and strengthening of existing buildings. Repair and strengthening methods for structural components such as walls, columns, beams, trusses, diaphragms, foundations, and connections are presented. The objective of such procedures may be either to repair a structure as nearly as possible to its pre-earthquake level of resistance, or to provide a structure with greater strength and/or ductility to resist the effects of future earthquakes. Throughout this Chapter it is assumed that post-earthquake damage in existing buildings has been evaluated, and that in the case of strengthening the desired level of earthquake resistance has been determined in accordance with Chapter 13, Systematic Abatement of Seismic Hazards in Existing Buildings.

14.1.1 SCOPE

A guide for collecting basic design information for determining appropriate repair and strengthening procedures is provided in Sec. 14.2. Repair and strengthening methods for steel, reinforced concrete, prestressed concrete, wood, masonry structural components, and foundations are described and discussed. Repair and strengthening of non-structural components such as parapets, ceilings, partitions, mechanical and electrical equipment, elevators, and light fixtures are covered in Sec. 14.9. A bibliography on repair and strengthening methods for materials and elements used in construction is also provided.

14.1.2 BUILDING CODES

In some jurisdictions building codes require, when changes in occupancy or additions or extensive modifications are proposed, that buildings be made to comply with current standards. Subject to local adoption of appropriate regulations, it is recommended that buildings be strengthened to the level set forth in Chapter 13. The fire resistance of modified structures must also be assessed to ensure compliance with relevant code requirements, since original fire resistance ratings may be diminished by repair or strengthening procedures.

14.1.3 ECONOMIC CONSIDERATIONS

The decision to repair, strengthen, or demolish a building is based on a variety of economic considerations. Rehabilitation costs may be justified by economic benefits such as increases in market value, anticipated lifetime, expected revenue, and/or possible tax or depreciation benefits. Therefore, a detailed program of repair or modification should not be formulated in advance of a conclusive economic evaluation.

Earthquake insurance may be a factor in considering a repair or strengthening program. Some companies may not insure structures considered to be poor earthquake risks, and when they discover that a building is a poor risk may cancel policies or demand high premiums with large deductibles. Most earthquake insurance policies will pay only for repairs that restore a building to its condition prior to damage, and any improvement above that level must be paid for by the owner.

When the decision is made to strengthen or demolish a building that has been declared hazardous, the economic impact on owners and occupants may be severe. Special governmental assistance or economic incentives have been introduced in some areas for selected buildings. In order to ensure equitable treatment, governmental jurisdictions should establish procedures for issuing legal notices and for initiating appeals when buildings have been declared hazardous.

14.1.4 MODES OF FAILURE

Failures may result from a variety of causes other than earthquake forces. Gravity load failures may be caused by foundation settlement, overloads on structural elements, or deterioration of materials. Failures caused by earthquake forces are most frequently found in components such as shear walls, moment-resisting frames, and horizontal diaphragms and their connections.

Structural and nonstructual components may fail due to poor anchorage or inability to accomodate story-to-story drift. Some common failure modes are listed below:

- Failure of roof and floor anchorage to walls.
- Failure due to pounding where adequate separation has not been provided.
- Failure where adequate reinforcement for shear and moment has not been provided at connections between concrete columns and beams, particularly where moment reversals occur.
- Failure of masonry where adequate reinforcement has not been provided.
- Failure of structural components caused by severe torsional effects, particularly in vertical resisting components where torsion in the horizontal plane greatly amplifies seismic effects.
- Failure of connections of girders to tops of columns and corbels.
- Failure of foundation soils from overturning loads, shaking, or liquefaction.

14.1.5 DESIGN OF MODIFICATIONS

The methods described in Chapter 13 should be used to assess the need for strengthening. However, more detailed information than is provided by those methods may be required to formulate a modification program, necessitating that material properties be determined by tests or detailed measurement. Basic modification concepts are as follows:

- Structures may be repaired by replacing or restoring damaged material.
- Individual structural components and/or their connections may be strengthened by methods such as increasing element thickness or size, adding reinforcement, and/or increasing the strength of connections.
- Structural systems may be strengthened by providing additional shear walls or vertical bracing. Additional columns may sometimes be added to reduce spans and, in the case of steel frames, connections may be changed from simple shear to rigid, moment-resisting connections. In all such cases the designer must ensure that a consistent load path or possible alternate load paths are provided within the modified structural system.

14.1.3 Cont.

- Upper stories may be removed to reduce the mass of a building.
- The period of the modified structure may be shortened and response characterisitics may be increased. Therefore, it is essential that the earthquake resistance of such modified structures be reevaluated.

An analysis of a strengthened structure may not provide definite results. The resistance to lateral-force producing phenomena can not always be completed accounted for by calculating the strength of the structural system only. Many such structures have undergone severe tests of capacity and demonstrated considerable inherent lateral stability. Increased lateral resistance provided by strengthening procedures generally will be highly indeterminate and quantitative values may have to be based in large part on engineering judgement.

14.1.6 QUALITY ASSURANCE REQUIREMENTS

Whenever repair and strengthening procedures are implemented, the minimum quality assurance should be the same as that required for new construction as set forth in Sec. 1.6.

The requirements for inspection and material testing for new work should also apply to modifications of existing structural components or systems; however, special procedures are necessary to assure the quality of alterations involving those techniques which are not used in new construction. Therefore, the overall sufficiency of a repair or strengthening program cannot be guaranteed by conformance of work to code and testing requirements for new construction.

14.1.7 PRESERVATION OF DESIGN AND CONSTRUCTION DOCUMENTS

Systematic programs for evaluating and repairing and/or strengthening existing structures are often hampered by the lack of original documents related to design and construction. Many structures could be repaired and strengthened much more efficiently and effectively if such records were available.

A repository for plans and other pertinent data used in designing and constructing all structures, other than single-family dwellings, for which a permit has been issued and used should be established wherever it is not currently in existence. To establish such a repository, an appropriate agency should be empowered by law to promulgate mandatory data acquisition. Pertinent information would be stored in a microfilm data bank integrated with a computer retrieval system. Each structure would be cross-referenced with a legal description of the property on which the structure is located so that a title search at property transfer would reveal a plan catalog number. County recorders would be provided numbers corresponding to the legal descriptions contained in the data bank.

The initial cost of establishing such a program could be met by state or federal grants or local assessments. Thereafter, sufficient revenue could be generated by a one-time charge included in the building permit fee and charges assessed when plans or other data were requested from the agency.

To ensure that plans and other data are not used improperly or illegally, regulations safeguarding the interest of past owners, architects, engineers, and others with a vested interest should be established. Re-use of plans without just compensation should be prohibited and, once plans have been issued, the statue of limitations for professional liability-in the event of re-use should automatically be invoked if it has not already expired.

14.1.7 Cont.

Information from soil investigations or other subsurface investigatory work could be used, under proper legal control, by researchers preparing subsurface maps, etc. These data would be a valuable by-product in the public interest, obtained at small additional cost.

Sec. 14.2 COLLECTION OF BASIC DESIGN INFORMATION

Buildings are seldom identical in design, or in degree of aging or deterioration, and usually have been designed under different code criteria. Variations in use, site conditions, and service exposure may necessitate different levels and procedures of repair and/or strengthening for similar structural components. The approach to acquiring basic design information is the same, however, regardless of the individual building differences or of the kind or magnitude of damage sustained.

A knowledge of code requirements and standards in effect at the time of design and construction is essential in developing a sound program of repair and strengthening. To facilitate collection of pertinent data, Table 14-1 is a procedural guide arranged in check-list form so as to serve as a basic tool for the process of quickly restoring facilities to productive use.

Sec. 14.3 STRUCTURAL STEEL COMPONENTS

Considerable flexibility in repairing damage to or strengthening of structural steel components is provided by highly developed techniques for bolting, welding, flame cutting, and flame strengthening. The selection of particular methods for repair or strengthening will be determined by specific conditions in each case. The principal factors which must be considered are: structural system, type of steel, prior service history with respect to damage or deterioration, and economic considerations such as cost and time required for carrying out the modification, and interruption of normal use of the structure.

14.3.1 PRE-MODIFICATION VERIFICATION OF MATERIALS

The mechanical behavior and properties of steel are primarily controlled by chemical composition and manufacturing process. Standard designations by the American Society for Testing and Materials (ASTM) set forth material specification and test procedures for ascertaining various properties. Steel for structural purposes is normally grouped in three general classifications: structural carbon steels, high-strength steels, and construction alloy steels. One or more of the properties listed below may be of significance in developing a corrective procedure:

1. yield point
2. yield strength
3. yield stress level
4. proportional limit
5. tensile strength
6. ductility
7. modulus of elasticity
8. strain-hardening
9. Poisson's ratio
10. shear modulus
11. weldability
12. machinability
13. formability
14. durability and corrosion resistance
15. fatigue strength
16. toughness
17. brittle fracture

18. notch sensitivity
19. impact strength
20. creep
21. relaxation

Research in manufacturing techniques and changes in design concepts, criteria, and specific use needs has resulted in the continuous development of new steels. Material used in older structures may no longer be in current use and therefore must be identified by reference to ASTM designations and specifications which were in effect at the time of construction. Once the ASTM designation and specifications of base materials have been identified, the pertinent properties, e.g., weldability, notch sensitivity, etc., may be considered in formulating a repair and/or strengthening program for damaged structural components.

Particular attention should be given to the effective stress capacity of members in service. A reduction for fatigue from past loading history and the effect, if any, of localized stress raisers must be considered. Loss of geometrical properties of sections due to corrosion from weather or chemical exposure and, in some cases, electrolysis must not be overlooked. Other section losses may arise from yielding under excessive load or fabrication processes such as punching, sheared edges, and weld undercut.

(A) NONDESTRUCTIVE TESTING. In addition to the general procedures outlined above, close inspection of each section in each structural component is of the utmost importance. An initial visual inspection will generally establish the need for and method of more detailed evaluation. The use of portable nondestructive testing equipment makes it economical to locate cracks and flaws in magnetic-type materials. A discussion of methods and comments on their limitations follows.

1. <u>Ultrasonic</u>. While ultrasonic testing is generally the most desirable for locating cracks and weld flaws, it is extremely difficult to obtain accurate results for fillet or partial penetration welds except when the testing is done by an extremely skillful operator qualified to interpret results based on type and size of weld. A determination of parent metal thickness is possible.

2. <u>Radiographic</u>. Gamma-ray testing is effective for penetrating up to an 8-inch thickness of metal, whereas X-ray testing is limited to a maximum penetration of 3 inches. Experience has shown that the testing of welded moment connections (tee joints) will not always provide meaningful results because of the geometry of the joint. Hence, care should be exercised in interpreting the film negatives.

3. <u>Magnetic Particle</u>. This method is primarily valuable in locating surface cracks and imperfections just below the surface of steel elements. An intense magnetic field is set up and magnetic particles are applied to the surface of a section under consideration. The exact size of cracks or subsurface discontinuities is thereby outlined.

4. <u>Fluorescent Magnetic Particle</u>. This method is especially useful in defining surface cracks, particularly in irregularly shaped members. Magnetic particle procedures are effective in locating discontinuities, weld cracks, lack of penetration, lack of fusion, and similar defects. The method has not had widespread use in new structural steel work, but could be useful in surveying possible damage to older structures.

(B) USE AND LIMITATIONS OF NONDESTRUCTIVE EXAMINATION METHODS FOR DETECTING WELD DISCONTINUITIES. The nondestructive examination of welds and weld-related material plays a very important part in assuring that a product meets its functional requirements. Each method of nondestructive examination has its abilities and limitations as outlined below.

Visual inspection is probably the most important and least expensive method of quality assurance, but it is limited to surface evaluation. Other methods of nondestructive examination must be supplemented with visual inspection for full quality assurance.

Ultrasonic examination provides a very effective three-dimensional method for use in the internal evaluation of weld metal, heat-affected zone, and parent material. This method is very effective in the evaluation of crack-type discontinuities, but rather poor for the evaluation of porosity or slag inclusions.

The usefulness of ultrasonic examination is related to material thickness in that it becomes more effective as the material thickness increases. Ultrasonic examination of material thickness less than 5/16 inch becomes very specialized and should be made only with the use of miniature probes. The AWS D1.1 procedure does not include the ultrasonic evaluation of material thickness less than 5/16 inch or the ultrasonic examination of fillet welds. Ultrasonic examination of fillet welds has some merit but it requires special instructions and procedures, especially for fillet welds smaller than 3/4 inch.

Radiography is a very effective means of internal nondestructive examination. However, the results do not exactly parallel those obtained with ultrasonics as radiography produces only a two-dimensional image. Radiography depends on loss of section in producing the subject image, making the method very sensitive to discontinuity orientation and displacement. These features make the method very effective for the detection of porosity and slag, while it is much less effective in the detection of flat-type discontinuities, unless they are in nearly perfect alignment with the exposing radiation, while detection of laminations is almost impossible with this method. The method is not usually considered to be applicable for the examination of weld joints other than the butt type.

Radiographic usefulness is also a function of material thickness but, unlike ultrasonics, its usefulness decreases as the material thickness is increased, making the process less desirable for evaluating welds in thicker materials.

Several classifications of radiographic film speeds can be purchased, making possible exposure of thicker material under equal exposure factors. However, as the film speed is increased, the resolution quality decreases, because of the increased film grain size which accompanies faster film.

There are two basic ways of producing radiation for the exposure of radiographic film, X-ray and gamma radiation. The X-ray is produced with electrical equipment which, in order to attain a unit which can be considered as being portable, limits steel thickness penetration ability to about 2-1/2 inches maximum.

The use of gamma radiation for examination of structural steel is usually limited to the use of two types of radioisotopes, Iridium 192 and Cobalt 60. Iridum 192 will penetrate steel thicknesses up to about 3 inches, while Cobalt 60 will penetrate up to about 6 inches. As the penetration ability increases, the quality of the radiograph decreases, making gamma radiography more difficult to interpret and less sensitive to discontinuity evaluation than X-ray, and Cobalt 60 less sensitive than Iridium 192.

Very strict safety measures must be taken with the use of radiography, as the radiation can be lethal. Three basic parameters must be considered with the use of radiation in reducing its lethal effect; they are time, distance, and shielding. Unless bulky permanent-shielded enclosures are used with these processes, considerable distance must be maintained between radiation source and workers and/or bystanders. The amount of shielding or the distance requirement also increases as the penetration capability increases.

Magnetic particle examination is limited to use on ferro-magnetic materials, and to the detection of discontinuities on or close to material surfaces. False indications can be produced by surface irregularities which might be misinterpreted as being weld flaws. The method is especially useful for the evaluation of fillet welds and weld bead or layer examination of multi-pass welds as the weld joint is completed.

Two basic principles may be employed in the magnetization of material:

1. The direct application of a magnetic field using a permanent magnet or a yoke which is an iron core wrapped with a solenoid coil, or

2. The induction of low-voltage electrical current into the test material using conductive prods of high amperage which form a circular magnetic field around the current path.

Four types of current may also be employed, A.C., half-wave rectified, full-wave rectified, and D.C. The use of A.C. current will restrict magnetic particle findings to flaws exposed to the surface, but A.C. current is best for detecting such flaws because the magnetic particles are readily excited during the testing process.

D.C. current will detect discontinuities which are below the material surface, but the magnetic particles tend to form poor patterns due to the lack of particle excitement during the current application. The advantages of both basic current types are gained with the use of half- and full-wave rectified current. Full-wave rectified current for magnetic particle examination is generally considered as being the most effective for general application.

Various colors of magnetic particles are available and are selected on the basis of contrast with the material surface. Paint does not usually interfere with magnetic particle examination except that it must be removed at points of electrical contact when using the current induction method.

The use of liquid penetrants is limited to surface evaluation. Penetrants are especially useful in the detection of tight surface cracks which might not be otherwise visibly detectable. Surface preparation for use with this method is usually critical.

Table 14-2 is a guide to the capabilities of various examination methods for detecting certain types of discontinuities in different types of weld joints.

The relative application costs of the various nondestructive testing methods can generally be classified as follows:

	Initial Cost	Operation Cost
Ultrasonic	4	3
Radiographic	4	4
Magnetic Particle	3 to 4	2
Liquid Penetrant	1	5

Cost: 1 = Low
2 = Medium Low
3 = Medium
4 = Medium High
5 = High

14.3.1 Cont.

(C) DESTRUCTIVE TESTING. It is generally not possible to identify the ASTM designations of all relevant materials without testing representative samples cut from existing sections. Even if the sections can be fully identified from nondestructive testing, it is advisable to verify pertinent material properties by laboratory testing.

(D) TESTING OF COMPONENTS. On occasion, the most expedient method of establishing the safety of a structure with respect to gravity loading is to test a complete or definitive segment of a floor or roof. A factor of safety must be established, usually twice the mandated code live- and dead-load, and a system devised of time-deflection and recovery measurements over a 24- to 72-hour period. If a structure when test loaded indicates no apparent distress and the residual deflection after unloading is within acceptable limits, the structure may be considered satisfactory for gravity loads. With the exception of individual frames, it is generally impractical to load test for lateral forces.

Floors may be uniformly loaded by using a water-ponding procedure which utilizes a plastic reservoir filled with water to a depth necessary to obtain the desired unit load. Concentrated loads may be applied by calibrated jacking devices, palletized materials, or drums filled with water or other materials with high specific gravity. Because roofs are sloped for drainage, loading is more difficult, but the procedures suggested for floors may be adapted. Similarly, individual frames or members may be test-loaded to a prescribed factor of safety.

14.3.2 REPAIR AND STRENGTHENING

In many cases, the seismic force resistance level of a structure should be increased even though the basic framing concept is sound. Often, such increases may be achieved by relatively simple procedures, depending on the deficiencies discovered during investigations of each frame member.

(A) MAINTAINING ORIGINAL STRUCTURAL SYSTEM. Joints may be upgraded by replacing existing fasteners, such as bolts or rivets, with higher-strength fasteners. Machine bolts may be replaced with high-strength ASTM A325 or A490 bolts. Similarly, existing rivets or high-strength ASTM A325 bolts may be replaced with higher-strength ASTM A490 bolts. Where conditions permit, holes may be reamed to allow installation of larger-diameter fasteners. The maintenance of proper edge and end distances is essential in such operations.

Connections may be welded to achieve an increased load transfer capacity. However, it is suggested that high-strength bolts not be used in combination with welds, since the welds should be designed to carry the entire force. Under some codes, existing rivets and properly installed high-strength bolts may be allowed to carry forces resulting from existing gravity loads, and welding need be adequate only to carry additional loads. Total replacement of a connection may frequently be the most expedient and economical method of improving a deficient joint.

In correcting deficiencies in members, it is vital not only to ascertain the mechanical properties of the material, but also to determine geometrical properties in order to permit necessary review of design calculations. Using the best available date of completion on a structure and field measurements of sections, the original section can usually be identified from the applicable AISC manual or from old catalogs of major steel producers. For very old sections, the publication entitled "Iron and Steel Beams, 1873 to 1952", available from AISC, is probably the best source for these data. If the exact section cannot be identified from a publication, a section nearly the same may be selected or as a last resort calculated properties from field measurements are generally sufficiently accurate.

14.3.2(A) Cont.

The manner in which a member is loaded under service conditions may dictate the means for increasing its capacity. In the case of compression members, reducing unsupported length, increasing the cross-sectional area, or replacing sections with higher-strength material may be used. Tension members, on the other hand, are usually strengthened by providing additional section area, or by replacing them with the same size sections of higher-strength material. Enhancing the capacity of flexural members usually depends on the variation and type of loading which greatly influence the selection of remedial measures.

A variety of welding techniques permit alteration of members in almost any manner short of complete replacement. However, each phase of member strengthening which results in an altered cross-section must be planned with a welding sequence in mind. It is desirable to maintain a symmetrical section whenever possible. The sequence of welding should be established so as to minimize warping and residual stresses. In order not to overstress the original section, the added material should be proportioned for a unit stress equal to the allowable unit stress in the original member less the gravity load stress in the original member. When new material is added to an existing section, new geometrical properties should be calculated.

Severely distorted members may often be restored by flame or mechanical straightening. Flame straightening requires a carefully controlled procedure and should be undertaken only by experienced personnel. With the straightened member as a base, increased capacity may be achieved by modifying cross-secitons as described above.

When enhancing cross-sections by weld procedures, the weldability of the base materials must be considered. Most major steel producers have publications available which specify the weldability of ASTM steels and provide information necessary to insure a satisfactory weld to base material. Similarly, modification of tension flanges in heavily loaded girders or beams requiring insertion of replacement pieces should consider the grain direction (with respect to rolling) and remaining flange material; the new piece must be oriented accordingly. The grain structure of structual steel plates can be a crucial factor in built-up sections, especially in tension flanges composed of different segments joined by butt welding.

Field considerations can significantly affect the cost of remedial procedures. Appreciable savings in cost may be obtained by using methods which minimize support shoring by arranging welding or oxygen-cutting procedures so that heating of a given cross-section will not prevent members from carrying gravity load. Occasionally, web stiffeners may be added in beams and girders. Fillet welds to tension flanges should be avoided if possible. When such welds are used, weld sizes should be carefully selected to reduce effects of stress concentration and possible local embrittlement. Removal of all extraneous material from steel surfaces, including paint, is essnetial in avoiding flaws in the repaired member.

The capacity of the base plate must not be exceeded when column load capacity is increased. While some increases in allowable bearing values for plates on concrete have been permitted in the provisions of ACI Building Code for Reinforced Concrete Buildings, 318-71, the stresses in the flexible plates using AISC criteria should be verified. If the plate thickness is insufficient, addition of stiffeners may be advisable. Engineering judgement and a realistic, practical approach are necessary in evaluating the compliance of base plates with the requirements of current codes.

(B) MODIFICATION OF STRUCTURAL SYSTEM. A variety of factors, including structural, architectural, economic, or societal needs, may preclude preservation of the original structural system with appropriate strengthening. In such cases substantial modifications of the structural system may be necessary to accommodate changes in the use of the building, new loading patterns, or specified increased level of earthquake resistance.

The process of integrating an original structural system with new structural components, such as additional members, frames, or walls, and replacement of floor or roof systems, is similar to that entailed in developing a new design. An analysis of the modified system will require investigation of stresses and deformations under gravity load as well as under combined effects of gravity and lateral forces. Any deficiency in the original components of the system would require strengthening with due regard to all the factors described in the preceding section.

A variety of modifications may be introduced. Wood floor systems may be strengthened or replaced by steel decking or reinforced concrete slabs. Lateral bracing may be strengthened or replaced by reinforced masonry or concrete shear walls. In such cases connections of the new walls to the floor must provide for transfer of the design lateral loads.

Addition of knee braces to frames designed solely for gravity loading affords a measure of lateral load resistance. When knee braces are introduced, columns should be checked for combined axial and bending stress. Also, beams must be checked for knee-brace-induced forces and if necessary strengthened.

Introduction of new elements or stiffening of existing elements to resist lateral forces may significantly alter the earthquake response of the resulting structure. Larger lateral forces may be induced in stiffer buildings and these should be taken into account in the modification design. Whenever possible, assymmetry in adding new components should be avoided because of torsional effects and possible increased lateral forces in critical components.

Sec. 14.4 REINFORCED CONCRETE

Damage in reinforced concrete buildings caused by differential settlement of foundations, accidental dropping of heavy equipment or other objects, local overloads, explosions, corrosion, deterioration, fire, and earthquake-generated ground motion have been repaired by special engineering service contractors. These special repair service contractors have broad experience in implementing repair techniques and, although much of their expertise is held proprietary, engineers are often able to utilize this experience in planning repair and/or strengthening programs for structures damaged during earthquakes. Recent laboratory test data may be utilized in conjunction with field experience in developing the most appropriate technique for mitigating hazardous conditions in structures.

14.4.1 PRE-MODIFICATION VERIFICATION OF MATERIALS

Methods of determining the condition of existing structures and material properties are discussed in Sec. 14.1 and 14.2. In the absence of as-built drawings and field construction data, coring of concrete to determine the strength of concrete and exposing and removing some steel reinforcement for visual-examination testing may be advisable in some cases. In taking cores and in exposing and removing steel reinforcement, special care must be taken not to reduce the load-carrying capacity of the structure. Where material is removed at a critical section, temporary shoring may be necessary. In other cases it may be less expensive to assume a minimum existing structure consistent with the codes and construction practices at the date of construction rather than to perform extensive field studies to determine necessary strengthening requirements and techniques. X-ray and Pachometer equipment can be used to verify the location of reinforcing steel in certain circumstances.

14.4.2 REPAIR AND STRENGTHENING

The materials available for repair and strengthening are discussed in this Section.

(A) MATERIALS. The more common materials used to repair or strengthen monolithic reinforced concrete construction are as follows:

1. Shotcrete. Shotcrete, known also as "gunite", is pneumatically applied concrete. It may be applied by a "wet mix" or "dry mix" process. Wet mix involves pumping of premixed cement mortar to a nozzle where compressed air impels it onto the substrate surface. To ensure a plastic mix either higher water content or special plasticizers are used in the wet mix. In some cases there may be difficulties in obtaining proper embedment of steel bars when using shotcrete. Where high water content mortar is used the shrinkage may also be high. Dry mix involves transporting premixed cement and sand by compressed air to the end of the hose where water is injected and mixed with the dry blend and the mortar is impelled against the substrate surface. While lower water content can often result in high-strength and low-shrinkage material, the quality of the work generally depends on the skill and workmanship of the men controlling the nozzle and the flow of water to it. The general practice of shotcrete application is described in the ACI Standard ACI 506-66 (reaffirmed in 1972), "Recommended Practice for Shotcreting".

2. Epoxy Resin. Epoxy resin is a general classification of adhesives manufactured from petroleum products and usually consists of two or more component chemicals mixed immediately prior to application. The various chemical compositions of these agents continually change to meet new performance requirements. For structural purposes, the material selected should contain 100 percent reactive solids. Detailed descriptions of epoxy resins and recommended uses are provided in manufacturers' catalogs and other publications. A procedure for verifying specifications and quality control should be established for selected materials. The viscosity, setting time, curing conditions, and mechanical properties of a mixture depend on the components of the mixture. The most common of these resins used in repairing building cracks are of the low-viscosity type, meaning that they can be mixed and pressure-injected into very small cracks. In order to fill spaces between cracked surfaces before the mixture has set, it is necessary that the epoxy not harden too quickly. Upon hardening, these materials strongly adhere to adjacent concrete and steel surfaces. Typical strength and stiffness properties of low-viscosity epoxy resins are given in Table 4-3 under "Epoxy - Neat".

Higher-viscosity epoxy resin mixtures can be used for surface coating or for filling larger cracks or holes. Epoxy-resin mixtures are highly toxic and the chemical reaction resulting from mixing the components is exothermal. The heat generated by the reaction can cause the mixture to boil if too large a volume of material is confined with no outlet for heat dissipation. Epoxy mixture strength is dependent on curing temperature and method of application. Curing temperature, time, and strength characteristics should be provided by manufacturers. Some components of epoxies deteriorate with time in storage, and some verification of epoxy properties used in the field may be advisable.

3. Epoxy-Mortar. For larger voids, it is possible to combine either the low- or high-viscosity epoxy resins with sand aggregates providing a heat sink and increasing the modulus of elasticity, see Table 4-3.

Epoxy-mortar mixtures have higher tensile strength, higher compressive strength, and greater shear capacity, but a lower modulus of elasticity, than Portland cement concrete. Thus, epoxy mortar is not a compatible-stiffness replacement material for reinforced concrete. Changes in the mechanical properties of epoxy mortar with large variations in temperatures must be considered when a large volume of replacement material is used.

Epoxy resin and mortar are combustible, and for most epoxies strength and stiffness decrease approximately linearly from values at about 70° F to zero at about 400° F. Therefore, to maintain required fire endurance, epoxy-repaired elements must be fireproofed when repair and strengthening are required. The probability of a severe earthquake occurring shortly after a severe fire is rather small, but the probability may be of greater importance when an earthquake is followed by a fire in a building previously repaired with epoxy materials.

A discussion concerning the applications of epoxy compounds in repair of concrete can be found in the ACI Report of Committee 503 entitled "Use of Epoxy Compounds with Concrete".

4. Fiber-Reinforced Concrete. A material that is stronger in tension than the original material can be obtained by adding steel, glass, or plastic fibers to normal or Type III Portland cement concrete. The fire resistance of the selected fiber concrete must be determined before use in repair and strengthening when repair or strengthening are required for gravity load conditions. A discussion concerning the applications of fiber reinforced concrete can be found in the ACI Report of Committee 544, entitled "State-of-the-Art report on Fiber-Reinforced Concrete".

5. Gypsum Cement Concrete. Gypsum cement concrete has not been widely used in structures. Typical properties of structural gypsum cement mortar are given in Table 14-3. Of the three materials listed, gypsum cement mortar has the lowest tensile strength, but its modulus of elasticity is close to that of concrete.

6. Portland Cement Concrete. Type III cement for mixing high early strength concrete has been used for many years. The properties of the concrete are described in appropriate ASTM specifications.

7. Quick-Setting Cement Mortar. Quick-setting cement mortar is a relatively new material patented by Republic Steel Corporation and originally developed for repairing reinforced concrete floors adjacent to steel blast furnaces. It is a nonhydrous, phosphate-magnesium cement with two components, a liquid and a dry aggregate, mixed similarly to Portland cement concrete. The cement mortar must be placed and cured in a water-free environment. Its properties are summarized in Table 14-3.

8. Preplaced Aggregate Concrete. This material, also known as "intrusion" or "grouted concrete", derives its name from the method of construction, in which forms are first filled with clean, well-graded, coarse aggregate and then mortar or grout is pumped into the void spaces. Intrusion mortars usually include expansive admixtures to ensure proper bond between aggregate and mortar. This material is well-suited for restoration work, especially where access is difficult or congestion of reinforcement makes shotcreting difficult.

9. Reinforcing Steel. Studies on anchorage of dowels and reinforcing bars in existing concrete have been limited. A common technique for providing anchorage is as follows: A hole larger than the bar is drilled and filled with epoxy, expansive cement, grout, sulfur, or other high-strength grouting material. The bar is then pushed into place and held until the grout is cured.

Field tests have shown that when the embedment length recomended in the ACI code is used the ultimate strength of the dowel or bar can be developed. In order to develop criteria for repair procedures, additional experimental data should be obtained to determine the minimum embedment length required to develop the tensile strength of bars which can be used with epoxy or high-strength grouting materials.

10. Mechanical Anchors. Mechanical anchors use wedging action to provide anchorage. Some anchors resist both shear and tension, while others resist only tension. The manufacturers of these mechanical connectors have specific recommendations for

installation and the strengths that will develop. Most of these strengths were obtained by static tests on selected materials. A limited number of dynamic tests have been conducted for specific applications. Anchor strength should be determined by an independent laboratory for specific applications, or code values with a large factor of safety should be used.

(B) REPAIR. Repair techniques must be selected according to the degree of damage and the level of repair to be accomplished. Two types of repair must be distinguished: (1) where damage is limited to moderate cracking, and sound reinforced concrete can be restored by filling in the cracks, and (2) where damage involves extensive cracking and spalling, and some of the concrete is shattered, but sound reinforced concrete can be restored by shotcreting or other means of replacing damaged concrete bonded to reinforcing steel.

1. Small Cracks. If concrete cracks are reasonably small (opening widths of less than 1/4 inch), the simplest method of repairing reinforced concrete elements is to pressure-inject epoxy. The procedure for epoxy injection is described below.

External surfaces are first cleaned of nonstructural materials. Plastic injection ports are placed along the surface of the crack on both sides of the member and secured in place with an epoxy sealant. The center-to-center spacing of these ports should be between 1 and 1.2 times the thickness of the concrete element. However, the spacing is dependent on the width of the element and whether or not pressure injection will be accomplished from both or only one side of a member. After these ports are in place, the surface of the crack between ports is sealed with epoxy sealant.

After the sealant has cured, a low-viscosity epoxy resin is injected into one port at a time, beginning at the lowest point of a vertical crack or at one end of a horizontal crack. Working at a port, the epoxy is pressure-injected until the material flows from the opposite side of the member at the corresponding port or from the next higher port on the same side of the member. When flow is observed, the injection port is closed and the equipment moved to the next port. After all ports have been used, the final port is closed and all ports remain closed until the epoxy is cured. This is normally a two-man operation, with epoxy injections occurring from one side.

Smaller cracks require higher pressure or more-closely spaced ports to penetrate the depth and width of the member with epoxy. Larger cracks allow larger port spacing, dependent on the width of the member. Epoxy injection is appropriate for all types of structural elements--beams, columns, walls, and floor units. Members repaired by this technique and subjected to loading conditions similar to those having caused the damage have shown failure cracks adjacent to epoxy repairs; i.e., the repair is stronger than the adjacent concrete material. However, the failure mechanism for the structure is not altered by the repair.

If there is a loss of bond between the reinforcing bar and concrete through a number of cycles of deformation, concrete adjacent to the bar is pulverized to a very fine powder and effectively creates a dam and prevents the epoxy from saturating the region. Pressure injection of cracks cannot restore bond. Also, cracks smaller than 0.003 inch may be difficult to pressure-inject effectively. Unrepaired small cracks and loss of bond result in a structural system less stiff than the original.

With full penetration of epoxy, original strength can be restored. However, recovery of only 70 to 80 percent of the original strength should be assumed. Because the member has been damaged, it is probable that the original section may not provide sufficient strength for the structure. Therefore, a technique for strengthening elements to avoid similar damage during subsequent earthquakes should be considered.

2. Large Cracks and Crushed Concrete. For cracks larger than 1/4 inch or for regions in which concrete is crushed, treatment other than by epoxy injection is required. Loose concrete should be removed, leaving only solid material. Material that is removed can be replaced with shotcrete, any one of special cement mortars, or concrete replacement. Selection of replacement material depends on the desired material characteristics as described earlier in this Section. When damage is severe, the need for additional shear or flexural reinforcement should be considered. If, however, development of the additional reinforcement into adjacent solid concrete regions is required, the repaired section will be stronger than adjacent existing material and failure in this adjacent section would be probable during a subsequent earthquake.

In the case of damage to wall and floor diaphragms it may be more economical to add new steel on the outside of surfaces and to cover this steel with concrete rather than to repair the damaged material. The increased weight from the materials must be considered in re-analyzing building forces and in checking the foundations.

3. Reinforcement. In a severely damaged reinforced concrete member, reinforcement must have buckled, elongated with excessive yielding, or fractured in extreme cases. Reinforcement can be replaced with new steel, using butt welding, lap welding, or in some cases by a splice. If practical, the repair should be made without removing the existing steel. The best approach depends on the amount of space available in the original member. Additional confinement steel should be added to inhibit future buckling of bars in this region. The additional steel will not substantially increase the strength of the member, but will extend its inelastic strength carrying capacity.

(C) STRENGTHENING. If the decision to strengthen or stiffen a building is made during the process of repair, a thorough analysis of the structural framing system must be made. The level to which a system should be strengthened or stiffened must be determined in accordance with the criteria established in Chapter 13.

1. Replacement of Structural System. Loading conditions must be evaluated at the time of strengthening and a new structural system selected as for a new building. However, particular care must be exercised in tying horizontal floor diaphragms into the lateral force-resisting system. Although the lateral-vertical load-carrying system has increased strength, if the connections to horizontal diaphragms are inadequate, this junction between the stiffened system and existing floor diaphragm will subsequently fail. The techniques discussed above for repairing and adding new materials to damaged regions are applicable. Foundations must be reviewed on the basis of the increased weight of the structural system.

2. Addition of New System to an Existing Structure. If new structural systems are added to the existing system, the existing internal stresses in members must be considered in analyzing behavior during subsequent overloads caused by earthquakes. Appropriate strengthening can delay failure or prevent structural damage from an earthquake. A smooth transition of stiffness and strength in the structural system must be provided similar to that for new construction. Increases in column and girder sizes can be accomplished by adding reinforcement adjacent to existing columns, by adequately confining steel, and by providing concrete cover for the additional width and depth of the member. Existing shear walls can be strengthened by providing additional reinforcing steel on the outside of the walls and increasing wall thickness with additional concrete. Care must be taken to anchor the ends of horizontal and vertical reinforcement into adjacent columns and beams in order to provide an integral wall unit.

In a recent investigation, the connection of reinforced concrete wall panels into existing concrete was studied. The types of wall panels considered were (1) a cast-in-place wall system tied into existing beams and columns with dowels lapping the wall steel and (2) large and small precast concrete wall panels connected to an existing frame with mechanical anchors. The cyclic strength, stiffness, and energy dissipation of these added walls were less than for the monolithic wall. However, the multiple panel precast wall sustained cyclic strength better than the others.

14.4.3 VERIFICATION OF MODIFICATION

The success of a repair process is difficult to assess because visual observations and minor mechanical probings and some testing were relied on to provide estimates of damage and to determine appropriate repair prodedures.

(A) INSPECTION. With good materials and field control it is possible to develop structural characteristics consistent with those assumed in the repair or strengthening program. The strength of the repair can be evaluated by standard testing procedures, using samples of materials utilized in the repair process. A strict field control and inspection process is necessary during the repair because the amount of material being used is less than that used in new construction and work is proceeding at strategic locations in the structure. Materials, field control, and inspection costs for repair will therefore be more costly than for equivalent new construction.

If repair materials are sufficiently stronger than existing materials and the strength variation of the repair materials is within normal tolerable limits, the lowest expected strength of the repair materials will exceed that of the existing material. Thus, the possibility that failure will occur away from the repaired region of a structure must be considered. When an existing system is strengthened, field control must be similar to that for repair work. In judging the strength of repair materials, preliminary tests of the original material must establish the necessary strength. In assessing the effectiveness of repair and strengthening processes, the evaluation must recognize the variabilities in construction operations and the limitations of prescribed field control.

(B) LICENSING. A program to license qualified contractors for repairing buildings is needed. For example, not every contractor has the experience and expertise necessary to perform effective epoxy injections. Therefore, a qualification list should be established so that building officials and owners may have confidence in the work done by contractors. Certification of inspection procedures should also be mandatory.

(C) LABORATORY DATA. When stronger materials have replaced original concrete and more confinement stirrups have been added around reinforcing steel, the repaired section will be substantially stronger than the original, and subsequent damage will probably occur outside the repaired region; for example, either the adjacent portion of the beam or connection will be damaged. Where the connection was repaired by introducing stronger material, failure could occur either in the column or the adjacent beams. Studies on the repair of beam-column subassemblages indicate that when a damaged beam end is repaired with a stronger material, a rapid loss of subassemblage strength and severe damage to the connection may occur when the connection has insufficient strength. The failure of a connection will form a hinge joint which effectively uncouples the intersecting members, and therefore the stiffness of the section will decrease rapidly, possibly resulting in stability problems. It is recommended that when beam ends are severely damaged, a significant amount of concrete be removed from the beam. Stronger material should be placed as far as is practical into the connection region as well as into the damaged beam end region.

Other repair techniques must be proven by laboratory tests. The cost of determining the characteristics of a particular repair technique is small when compared to the consequences of using inappropriate processes. Frequently, the cost of these tests will be recovered by savings in the repair procedures developed.

Sec. 14.5 PRECAST CONCRETE AND/OR PRESTRESSED CONCRETE STRUCTURES

Many procedures used to repair or strengthen cast-in-place concrete containing mild reinforcing steel also apply to precast and prestressed concrete construction. These procedures are covered in Sec. 14.4 and only modifications necessary to accommodate the unique requirements of precast and prestressed concrete are discussed in this Section.

14.5.1 PRE-MODIFICATION VERIFICATION OF MATERIALS

Nondestructive load testing per ASTM or local code requirements should be carefully controlled so as not to cause failure in a member which could otherwise have been repaired. Verification of material properties and existing capacity is important in determining appropriate test loads. Existing concrete may be sampled by cutting or coring to determine type, strength, and quality.

When concrete is removed by coring or cutting, the effect on the remaining section must be considered. This is especially important in prestressed concrete structures where the removal of a high percentage of a section could adversely affect subsequent attempts to repair an element because the section's capacity has been reduced. Also, prestress tendons under stress may fail if damaged by equipment during concrete cutting. Proper procedures and care must be used when working around tendons or their anchorages.

Prestress tendons may be exposed for a given length (approximately 10 feet) and stresses measured by instruments without damage to the tendons. In some cases samples may be cut where their removal will not prevent successful repair of the member. Steel properties can be determined by testing samples cut from the tendon or identified using manufacturer's markings. Unbonded tendons should not be cut. If, however, this should be necessary, the method illustrated in Figure 14-1 may be used.

14.5.2 REPAIR AND STRENGTHENING

Precast or prestressed concrete members can be repaired or strengthened by a number of procedures as discussed below.

(A) PRECAST CONCRETE MEMBERS. Thin wall panels (architecturally precast) can be strengthened by (1) epoxy injection of cracks, (2) adding structural steel members either to the interior or exterior of panels, (3) replacing damaged concrete sections, or (4) replacing damaged panels.

Structural members can be strengthened by (1) adding reinforced concrete as required and permitted by the capacity of the structure to carry additional load, or (2) addition of post-tensioning. In adding post-tensioning to an existing structure, the effect that this force may produce in the structure due to creep, bending of columns, changes in shear stress, etc., must be considered.

(B) PRESTRESSED CONCRETE MEMBERS. Prestressed concrete members (pretensioned or post-tensioned) should be replaced by the same basic materials using methods for cast-in-place or precast reinforced concrete with some important exceptions.

Prestressed concrete design is based on controlling concrete stresses by the magnitude and location of axial force produced by the prestressing steel. Any change in this induced stress level will alter the capacity of a section to carry load.

Repair procedures must consider that a repaired or replaced concrete section may no longer carry any prestress force, and may thus not contribute in the same way to overall section resistance to load. Before beginning repair procedures, concrete sections may need to be shored or loaded so that repaired sections when completed will contribute their share of strength to the total section.

When a concrete section or entire member is removed for replacement, the continuity of adjacent framing may be disrupted, tendons may lose bond, and controlled stresses in the section due to prestress may be relieved. The effect of such removal on adjacent framing must be carefully considered.

In some ungrouted post-tensioned systems damaged prestressing steel can be replaced when contained in a conduit. Most prestressing steel is, however, embedded, grouted, or otherwise installed in such a manner that removal is difficult.

By placing prestress tendons external to a damaged member (concrete or structural steel) and subsequently stressing these tendons over a certain profile, additional support and load-carrying capacity can be provided. Tendons can be covered with cast-in-place concrete or pneumatically placed concrete for protection (See Figure 14-2).

Concrete surrounding post-tension bearing plates can be repaired by epoxy injection or grouting. To provide final satisfactory bearing capacity, it may be necessary to release the load temporarily. Methods are available for testing the force in prestressing steel contained in a prestressed member. In unbonded post-tensioned members, a jack can be attached to the end of the tendon and a lift-off measure of forces can be obtained. Stress in exposed steel can be measured when the tendon has been freed to vibrate over a specified length.

(C) PRECAST CONCRETE CONNECTIONS. The behavior of a connection after repair, whether fixed or flexible with respect to the structural frame, should be the same as that originally provided in the structure.

Temporary support of precast elements during repair may pose special problems for architectural panels attached to the exterior of a building, or for other structural members required to support shoring.

Techniques for repairing damaged connections are (1) epoxy injection or grouting, (2) removal and replacement, (3) addition of reinforced concrete or steel elements with drilled inserts, dowels, keyways, etc., or (4) welding of the connection hardware; overheating of steel embedded in concrete must be avoided since expansion and spalling of concrete in the area of the connection may occur.

14.5.3 VERIFICATION OF MODIFICATION

Procedures for verifying the repairs and strengthening made to precast and prestressed elements and structures are similar to those for reinforced concrete (see Sec. 14.4.4).

Sec. 14.6 WOOD

Wood and wood products considered in this section are sawn lumber, glued laminated timber, plywood, and timber poles and piles. Material properties and structural characteristics are reflected in assigned grades that depend on natural factors influencing internal structure and processes used to manufacture specific wood products. Factors that determine assigned grade are described in standard specifications and general references such as the following: Grading Rules for Western Lumber; Official grading Rules for Eastern White Pine, Norway Pine, Jack Pine, Eastern Spruce, Balsam Fir, Eastern Hemlock, and Tamarack; Southern Pine Inspection Bureau Grading Rules; Standard Grading Rules for Canadian Lumber; Standard Grading Rules for Northeastern Lumber; Standard Grading Rules for West Coast Lumber; Standard Specifications for Grades of California Redwood Lumber; U.S. Product Standard PS 1-74 for Construction and Industrial Plywood; Standard Specifications for Structural Glued Laminated Timber of Softwood Species, AITC 117-77; Wood Structures - A Design Guide and Commentary; Wood Handbook of the U.S. Department of Agriculture; and the National Forest Products Association National Design Specification for Wood Construction.

Buildings with wood frame walls, floors, ceilings, and roofs have generally performed well in earthquakes. Such performance is attributable to wood's relatively light weight and ability to withstand dynamic overloads without failure. Earthquake failures that do occur in wood-frame buildings are usually in the form of permanent deflections or failure in connections.

Engineered timber buildings--those other than dwellings of wood-frame construction --are more susceptible to damage under seismic forces. Such buildings usually consist of a series of single members with long spans that depend on columns for support. Members that support relatively large areas are normally much more critical to a building's structural integrity than are individual members used in conventional stud wall framing systems. In the event of distress due to an earthquake, the quality of the individual engineered timber members and fastenings must be evaluated and repaired in accordance with code provisions.

14.6.1 PRE-MODIFICATION VERIFICATION OF MATERIALS

In order to determine the quality of materials and the adequacy of construction details with respect to code provisions, finishes on members must frequently be removed. The decision to undertake extensive explorations involving removal of finishes should not be made without weighing the benefits gained thereby against the considerable costs of such exploration. A specialized knowledge of wood species and lumber, plywood, glulam and round timber grades, and of appropriate design values as mandated by codes and standards is required in order to determine the quality of existing materials. When grade marks are distinguishable, the task of such determination is simplified since the grade can normally be related to a recommended design value by reference to the National Design Specification for Wood Construction or other relevant document. Where grade marks are not discernible but such information is essential, persons with experience in identifying and grading wood products should be engaged to evaluate the quality of materials and their strength properties in place. Grade trademarks on plywood panels may be used to identify the standards under which the panels were manufactured. Unmarked panels may require examination by persons trained in identifying wood products in order to establish structural capacity.

In some instances, it may be advisable to increase the resistance of buildings against seismic forces through design modifications. Occasionally, timber may have deteriorated and require replacement. Conditions that may require removal and replacement include adverse moisture content and temperature, decay, insect infestation, chemical attack, excessive checking or splitting, fire damage, delamination of plywood and glued laminated timbers, and corrosion of fasteners.

(A) LUMBER. The following factors should be considered in establishing physical and mechanical properties:

1. Moisture Content. The initial cross-sectional areas of wood members are reduced as the moisture content is reduced; however, most strength properties increase. The increase in strength properties associated with a reduction in moisture content usually more than compensates for any reduction in cross section. Connections may loosen due to shrinkage, particularly timber connectors or bolts, with a possible corresponding reduction from full design value. Such connectors should be tightened to re-establish joint integrity. At temperatures in the vicinity of 68°F, clear wood property changes are as described by the following relation:

$$P = \left(P_{12}\,\frac{P_{12}}{P_g}\right)^{-\left(\frac{M-12}{M_p-12}\right)}$$

where P is the property, M is the moisture content (not to exceed Mp), M_p is the moisture content at fiber saturation (varies by species, but is generally 25 percent), P_{12} is the property value at 12 percent M, and P_g is the property value at green condition. Design values published in building codes have already accounted for the effects of moisture content on size and strength.

2. Temperature Effects. As wood is cooled below a temperature of 68°F, the strength increases and, as the temperature rises, the strength decreases. The temperature effect is immediate, with the magnitude depending on moisture content. The immediate effect is reversible up to a temperature of 150°F, with affected members recovering essentially all strength when the temperature returns to normal. Prolonged heating to temperatures above 150°F will result in permanent loss of strength.

In some regions, structural members are periodically exposed to elevated temperatures. Where the relative humidity normally accompanying elevated temperatures is low, wood moisture content is lowered and the immediate effect of exposure to elevated temperature is mitigated by this dryness. However, where high temperature is accompanied by high relative humidity, the combined effect of these conditions will result in strength reduction. Building code design values are traditionally based on ordinary temperature fluctuations and occasional short-term heating to temperatures to 150°F.

When wood structural members are cooled to extremely low temperatures at high moisture content, or heated to temperatures of 150°F or greater for extended periods of time, design values should be adjusted. Approximate average factors are given in Table 14-4 and can be used as a guide in making such adjustments.

3. Mold, Fungus Stain, and Decay. When the moisture content of untreated wood exceeds 19 percent and the wood is exposed to oxygen in a temperature range of 50°F to 90°F, mold, fungus stain, and/or decay may develop in wood elements. Wood in confined air spaces, or in direct contact with earth and masonry or concrete in contact with earth is especially vulnerable. Wood-to-wood or wood-to-metal contact surfaces are also vulnerable if moisture or high humidity is present. Increment borers--T-bar-shaped coring tools which produce a 1/4-inch-diameter core that can be visually inspected to determine the depth to sound material--may be used to determine the extent of damage.

Some types of fungi may cause a reduction in wood strength. Generally, affected wood should not be used where strength is a factor. However, if such material is used, design values should be modified in relation to the type and amount of fungi evident in order to assure safe strength values.

4. Insects. Wood-destroying insects often infest wood products. Damage frequently occurs below the surface of the material and is therefore not readily discernible. Subterranean termites are the most common of these wood-destroying insects and, since they almost always nest in the earth, usually infest wood that is in contact with or in close proximity to the earth. Dry-wood termites are less common but do exist along Southern coastal regions of the United States. Infestation may occur at any point above the ground. Beetles do less damage and are relatively easily controlled. Where insect damage is suspected, a pest control operator should be engaged to identify the kind and extent of infestation and to eliminate any infestation before repairs are completed.

5. Chemical Degradation. Wood is resistant to many chemicals such as mild acids and solutions of acidic salts, although some chemical solutions may affect wood by causing a reversible swelling or irreversible changes in the wood structure due to reaction with its chemical constituents. Alkaline solutions are more destructive than acidic solutions, and hardwoods are more susceptible to attack by both acids and alkalis than are softwoods.

Fasteners and metal connectors may corrode as a result of the combined effect of moisture and chemicals. Such corrosion can be avoided by eliminating the source of moisture or by using corrosion-resistant fasteners.

Chemicals formulated for the purpose of treating wood against deterioration by fungi or insects do not usually result in strength loss, but pressure impregnation with fire retardant chemicals may cause a loss of strength. Thus, in the case of chemical treatment for fire retardation, a reduction is generally required by the code. Metal fasteners in contact with moisture and chemicals may also be adversely affected.

6. Shakes and Checks. The shear resistance of a beam is a function of the shearing strength of the species and the extent of shakes or checks. In a checked beam, however, shearing strength is more uniform over the unchecked portion than in an unchecked beam, explaining why a checked beam will carry a load for which it would appear to have insufficient shear strength. Checks and splits are especially serious in connection areas.

7. Fire Damage. Earthquakes may cause fires that damage structural wood members. Wood subjected to temperatures in excess of 350°F will decompose, leaving a charcoal residue; wood rarely burns directly. In common with other organic material, decomposition of wood causes emission of flammable gases and tars. Untreated wood generally chars at the rate of 1/30 to 1/50 inch per minute or about 1-1/2 inches per hour under standard fire test conditions. Because heat penetrates extremely slowly ahead of the char, the temperature 1/4 inch ahead of the charring wood will be only about 360°F. Consequently, sizable wood members generally maintain a high percentage of strength after exposure to fire conditions. Wood sections that have not been exposed to such high temperatures will retain original strength properties. The depth of damaged wood may be easily identified by the line of dark-colored char. The insulative qualities of wood protect glue lines of glued laminated members just as they protect the inner areas of solid-sawn timber. Plywood panels in which veneers show signs of char over a substantial portion of their area should not be depended on for strength. Where panels are discolored or show signs of scorching, glue line integrity should be investigated. Exterior-type adhesives are generally unaffected by heat, but some interior-type adhesives are adversely affected. Glue lines may be evaluated by selecting and testing representative panels in accordance with U.S. Product Standard PS 1, 4.5.4. Functional property values of charred sawn lumber, round timbers, glued laminated timbers and plywood should be based on the remaining net section of uncharred wood.

(B) GLUED LAMINATED TIMBER. Strength-inhibiting factors for glued laminated timber are essentially identical to those for sawn lumber and timber, except that glued laminated timber is generally free from checking. Such members are constructed from laminations thin enough to season readily prior to lamination without developing checks. Since checks lie in a radial plane and the majority of laminations are flat grain, checks in the laminations of horizontally laminated beams are so positioned that they will not materially affect shear strength. If members are designed with vertical laminations and detrimental checks are present, their effect should be evaluated as for sawn lumber. Checks in flexural members only affect horizontal shear, which is usually not of particular significance unless checks are extremely deep and shear governs the design of the member. For design purposes, shear strength reduction is assumed to be directly proportional to the ratio of depth of check to the width of the flexural member. Column checks are not significant unless a split develops which in effect causes a reduction of l/d in the member. In general, the safety factor is such that minor checks may be disregarded.

Field investigations may indicate that other factors require that design values be reduced, such as:

1. Moisture Content. Concave surfaces of individual laminations in the depth of a member associated with checks at the top and/or bottom of the concave surface may indicate a large difference in moisture content in adjacent laminations at the time of manufacture. Significant moisture differences in regions where shear is important may necessitate that allowable shear strength be downgraded, with the reduction to be based on engineering judgement considering member importance.

14.6.1(B) Cont.

2. End Joints. Openings visible on either side of an end joint may indicate that a joint no longer complies with code requirements. The extent of unsound end joints may be determined by probing with a 0.004 to 0.006 inch feeler gauge blade. For finger-type end joints, the difference in length of tip opening on each side of the member or variation of the tip opening across the member width on the top and/or bottom should be similarly determined. In assessing the importance of a loss of joint value, the location of the joint in any given member must be considered.

3. Glue Line. Openings may occasionally be visible at glue lines, resulting from inadequate bond or checks and shakes. Normally a visual inspection of both surfaces of an opening will reveal the cause. When in doubt, easy movement of a feeler gauge in the glue line will indicate a glue bond deficiency which may be verified by testing 1-inch cores taken at right angles to the glue line in low stress locations.

14.6.2 REPAIR AND STRENGTHENING

Engineering methods of analysis and procedures for design of repair or strengthening for existing wood members and frames are identical to those for new construction. It is, however, important to determine the effect of deficiencies with respect to current code provisions for strength and other mechanical properties after evaluation of the effect of individual components on the behavior of a structure as a whole, especially after consideration has been given to variations in stress and loading resulting from notches, holes, and knots in critical areas, and connections adversely affected by shrinkage.

Individual members or components may not conform to code requirements due to penalties necessitated by in-service or continuing-service conditions. Similarly, connections that transfer continuing service loads may no longer have sufficient capacity to conform to code provisions. As the state-of-the-art of earthquake design progresses, seismic requirements may be altered and elements may have to be strengthened to meet current requirements.

The method selected to repair or strengthen elements is generally based on economic considerations. Replacement of an element determined to be nonconforming to code requirements is the most direct approach, especially if connections at the point of load transfer are also below required capacity. In such cases, reference should be made to Chapter 9 for criteria relating to wood construction for new work.

(A) CHECKS, SHAKES, OR SPLITS. Where the maximum end distances for connections do not prevail and/or checks or splits are in or adjacent to the joint area, stitch bolts or clamps may be used as reinforcement provided the joint is kept permanently compressed. Steel package strapping may be used to clamp the checked or split member at the joint. Openings affecting shear strength may be repaired using a two-stage "heal and seal" process with epoxy glue. Exposed surfaces must be devoid of foreign matter that would inhibit bonding of epoxy to the wood. The procedure may be ineffectual where wood has been subjected to more than minor changes in moisture content. In any case, manufacturers' requirements and limitations must be carefully noted. Fire resistance of elements and structures repaired with epoxy must be assessed to ensure compliance with relevant code requirements, since the original fire resistance ratings may be diminished by such a repair or strengthening procedure.

Dowels may be used in some cases to increase shear capacity for a specific member provided that the cross section as separated by checks or splits is compatible with dowel size and desired spacing. The net section must also be capable of resisting other types of stress occurring at a given location in accordance with relevant code provisions. A decrease in available section often results in an overstress in either tension or for flexural members in tension parallel to the grain. This overstress can be overcome using localized prestressing employing rods in tension as in concrete post-tensioning.

14.6.2 Cont.

(B) INADEQUATE CROSS SECTION. Where conditions permit, the cross section of tension members may be increased by nail-gluing two-inch lumber to members with an exterior-type void-filling glue. The moisture content of sections being joined must be approximately the same and load eccentricity must be avoided. Since the integrity of the glue line should be verified by core tests, provision must be made for this loss of section in the area of added material.

Prestressing may be used to offset the need for a larger cross section, utilizing connections at either side of the member for the rod or cable anchorage. In some cases, steel angles, channels, rods, or cables may be used to transfer the entire load with the wood section in-place acting as a form of lateral stabilizer. With either method, the steel may require some form of fire-proofing so as to avoid a downgrading for purposes of fire rating of a structure classified as heavy timber.

Cross-sections of flexural members may be built up by using a gluing procedure similar to that described for tension members in which all aspects of stress are considered. Occasionally, a composite beam of steel and wood cross section can be developed, especially for lightly loaded short-span beams or purlins.

Short columns may be repaired by a "heal and seal" process or by cinch bolts and/or clamps as previously discussed for repairing checks, shakes, or splits. The load capacity of an intermediate or long column may be increased or repaired by reducing the slenderness ratio of the member by creating a spaced column with the addition of another piece. The location of spacers must be carefully considered. A tee or H-section can be created when fasteners complying with code provisions are provided and the load is properly distributed over the newly created cross-sectional area.

Cross sections with capacity reduced by insect damage or decay must be reinforced to carry load across the affected region. The decay zone must be treated with a wood preservative, usually by pressure injection of a preservative through predrilled holes at intervals in and adjacent to the decayed area.

(C) WOOD DIAPHRAGMS. While sheathing is often adequate to develop design shears, the nailing available to transmit this shear may be inadequate. Blocking may have been omitted and may be necessary. A renailing program based on the calculated shear distribution in the diaphragm may be the most expedient solution. If there is concern over potential splitting of frame members, box nails and staples may be used, providing that slip values for these fasteners comply with relevant code provisions.

Many older structures have straight-laid board diaphragms, attached by nail couples that may no longer comply with code requirements. Since allowable shears for such diaphragms are based on nail couples, it may be necessary to install a plywood or diagonal-sheathed overlay designed to transmit the entire calculated shear as a new diaphragm. The plywood may be laid with its face grain at a 45° angle to the direction of the boards. It is important to ensure that all plywood edges are backed by a nailing surface and do not coincide with the original board deck. All aspects of load transfer to vertical resisting elements must be considered and the system made to conform to code requirements. A horizontal bracing system may have been used as the primary lateral force distributing element with the existing wood sheathing assumed to carry gravity loads only. If such a system does not conform to current code requirements, a new plywood diaphragm overlay designed to carry the total shear anticipated by the analysis may be added. Little, if any, shear should be assigned to an existing horizontal truss system because the sheathed diaphragm would then be considerably stiffer and would carry virtually all shear.

When overlay plywood sheathing is installed, joints should be staggered to avoid end bearing on the supports of the existing sheathing. Pre-drilling of nail holes even though costly may be advisable to preclude splitting of dry bearing members.

1. Wall Ties. Anchorage of diaphragms to shear walls in older structures may not conform to code provisions. In many structures with masonry or concrete shear walls, anchorage is provided through edge nailing of the diaphragm to 4-inch thick wood ledgers bolted to the walls. Studies of damage to such structures in recent earthquakes indicate that this is a poor method of anchorage. Such anchorage may be improved by installing steel-strap ties fastened to walls through the existing ledger bolts and then either to primary or secondary vertical load-supporting members in the diaphragm assembly. Additional nailing of the diaphragm sheathing to the primary or secondary member to which the tie is attached may be necessary to distribute design forces normal to the wall in accordance with code provisions.

In some cases, new ties may be installed by fastening a steel strap to a wall with self-drilling cinch anchors capable of accepting the design force normal to the wall and then to the diaphragm as previously described. Any number of innovative methods of creating a positive wall-to-diaphragm anchorage which meets code requirements may be devised.

2. Shear Bolting Deficiency. When diaphragms are upgraded, shear transfer to walls may no longer conform to code requirements. For masonry or concrete walls, self-drilling shear-resisting anchor and bolt assemblies installed through otherwise-satisfactory existing ledgers may be used to bring a system into compliance. Where new ledgers must be installed, additional shear bolting may be provided by dry-packing bolts in oversized holes or by using anchor and bolt assemblies as described above. Other satisfactory solutions to the problem of shear bolting deficiency may be determined by field considerations.

(C) SHEAR WALLS. A wood shear wall must transfer both lateral forces imposed by the horizontal distributing element through shear to lower levels and vertical forces generated at ends by overturning moment. Tie-downs at each floor level where uplift may develop at the ends of shear walls may not have been provided. An anchor capable of resisting the design concentrated load at wall ends should be installed so that code provisions are met. Such installation poses no special problems. The receiving elements at the lower level of such walls, and the path of forces through these elements and into the ground, must be evaluated for load-transmitting capability and, where inadequate, made to conform to code provisions. Many types and combinations of sheathing for increasing the shear capacity of walls have been assigned safe shear values in buildings codes. Occasionally, only additional nailing is necessary to bring the capacity of such walls to a level conforming to prescribed code capacities.

Where existing diaphragms are overstressed, one or more vertical load-resisting elements or shear walls may occasionally be added in strategic locations, creating several smaller diaphragms. Shear in the respective smaller diaphragms and in existing shear walls may thereby be reduced to a level conforming to code requirements. New design criteria would apply to such a situation and to any repair or strengthening technique used. New wood shear walls should not be introduced in structures in which masonry or concrete construction contribute to the lateral force to be resisted without referring to the prevailing building code of jurisdiction.

(D) CONNECTIONS. In wood-to-wood connections, edge distance, end distance, and spacing will occasionally permit a larger diameter bolt to be installed in an existing lap joint, thereby improving load transfer capacity. A lap joint may be improved by inserting a relatively thin steel plate between two wood surfaces of sufficient length to permit bolts to be added beyond the ends of the joined pieces. Similar procedures, using either steel splice plates with wood fills or wood splice material with wood fills, may easily be developed where conditions permit.

High-strength adhesives may be used to improve the strength of connections in some cases. Contact surfaces may be coated with an exterior-type void-filler glue and existing bolts used to provide the necessary clamping pressure on the glue line during the curing period.

Steel plates may be used to strengthen splices not conforming to code requirements in connections of wood-to-wood or wood-to-steel. Field conditions will differ, but no new design techniques are involved.

In recent years, standard steel accessories have been developed for virtually every wood framing condition, including joist or beam hangers, glulam saddles and hinge connectors, post caps and seats, post bases and tie-down fittings, framing anchors, sill anchors, corner brackets, angle gussets, wall bracing and bridging, and other special fittings. Additionally, nailable truss plates, shear plates, split rings, clamping plates, toothed rings, etc., are available and may be used to develop connections. Information and data are provided in manufacturers' catalogs which are readily available to the construction industry.

Sec. 14.7 MASONRY

Masonry assemblages encountered in a repair program may be grouped according to the classifications in Sec. 12.A.3 as listed below:

Unburned clay masonry	Hollow unit masonry
Stone masonry	Partially reinforced masonry
Solid masonry	Reinforced gypsum concrete
Cavity wall masonry	Gypsum masonry
Grouted masonry	Glass masonry

Older masonry construction may deviate in detail or material requirements from these classifications and local practice may also result in different classification.

Overstressing caused by seismic loading may cause different types of damage to masonry walls. Cracks may be difficult to detect when they have occurred in the elastic range and have closed after the earthquake. By studying a structural system before making a specific investigation, crack locations may be anticipated and evaluation of damage expedited. Some of the typical damage in masonry construction is discussed below.

14.7.1 NONBEARING MASONRY WALLS

Nonbearing masonry walls are used to separate space, including the exterior from the interior of a structure. These walls are generally not intended to carry vertical load other than the weight of the wall.

In-filled walls consist of masonry wall units constructed between columns, floors, and beams. These walls frequently function as shear walls bounded by structural elements, although they may not be designed for that purpose. In older construction, in-filled walls are frequently not tied to beam soffits nor to columns, although occasionally there are dowels in floors which were used to anchor the walls to the floor.

Since these walls are stiff and are placed between structural elements they resist deformation of the frame and if loads are sufficiently great they will fail. For loads in the plane of the wall, the common modes of failure are:

1. local crushing of corners
2. bed joint sliding
3. diagonal cracking through the head and bed joint pattern
4. diagonal cracking through the units

Walls may also form wedge-shaped portions while providing bracing and restraint to columns, possibly causing secondary failures in frame members.

In the absence of anchorage at the edges of in-filled walls forces perpendicular to the plane may cause the walls to displace laterally. However, many in-filled walls perform adequately in a direction perpendicular to their plane when they are between columns or between very stiff floors and ceilings because they will arch if there is a bond or key at the ends adequate to prevent such displacement. These walls may, however, crack at the ends and at arch points; i.e., at the effective abutment and crown.

Other nonbearing walls or partitions may rock on their bases, forming horizontal cracks at the bases which are difficult to locate because of residual compression. These partitions may also function as shear walls when bounded by frame elements.

Nonbearing walls may fail through in-plane vertical splitting and thus lose the face shells of hollow units. Face shells may fall off or merely crack, leaving slight bulges or hollow spaces. Similar splitting failure may occur in two-wythe solid masonry.

Load-bearing masonry walls carry the vertical load of the structure, either concentrically or eccentrically. In a building subjected to lateral seismic forces, these walls must also resist forces which may be either perpendicular or parallel to the plane of the wall. When such walls carry heavy vertical loads, the compression reduces the tendency of the wall to slip or displace laterally when loaded perpendicular to their plane. Some arching may result causing crushing at the edges of the supports. The crushing may be accentuated by the combined effects of compression and bending.

Types of damage that occur to bearing walls are similar to those for in-filled walls: failures of connections tying the wall into the structure, and failure perpendicular to the wall causing either horizontal cracks near the center of the wall (arching from top to bottom) or causing vertical cracks (arching between supports at the ends, such as columns or pilasters). In addition, walls may fail due to shear forces in the plane of the wall.

Shear walls provide lateral rigidity to structural systems if proper connections are made and resist lateral forces. In addition, they may or may not carry vertical loading. If shear walls are overloaded, diagonal cracks due to diagonal compression or tension may appear, and failure either through the pattern of the joints or diagonally through the masonry units may occur. Local compression failures at the corners due to diagonal racking of a frame, sliding on the base (or top if connections are inadequate), or bed joint sliding may occur.

Piers are portions of walls between openings in which vertical and lateral loads may be concentrated. Piers may be restrained as cantilevers pinned at the top, as cantilevers fixed at top and bottom, or may be pinned both at top and bottom. Damage will differ for the different restraint conditions. Piers will occasionally fail in bending due to loads perpendicular to their plane as they span from top to bottom, with horizontal cracks at the center. They may function as shear elements in a load-bearing shear wall with the typical diagonal X-cracks commonly noted in piers and, as stated above, they may fail due to shear loads either in bending or in shear, or in combination.

During an earthquake there may be tension yielding first on one side of a pier and then on the other with compressive stresses concentrated at opposite corners. This compressive stress when added to the shearing stress may cause local failures, and when subjected to further cyclic loading degradation may occur.

Failures due to bending moment such as tension failure or overstressing of inadequate steel reinforcement may or may not be serious. Horizontal cracks will appear as the steel elongates, but only further evaluation will reveal whether the steel has been stretched beyond its elastic limit. Horizontal cracks which may have occurred at the top or bottom of a pier may have closed due to the compressive load on the pier.

Lintels are the portions of walls over openings. The load assumed to be carried by the lintels are subject to considerable judgment. In general, masonry material will arch over an opening if there is adequate abutment reaction, as demonstrated by jack arches. The load on lintel elements is generally small.

Masonry elements normally are not used as beams, and failures will probably occur at the ends of units, possibly due to end rotation at the support. Such beams are not generally part of a rigid frame, so that the X-cracks from diagonal tension near each end generally do not develop.

Column elements are occasionally isolated or they may be considered as portions of walls for carrying vertical load, or built integrally with the wall for carrying vertical load as pilasters or to provide buckling stiffness. Such columns are most frequently damaged at the top due to beam end rotation and concentration of load. Until recent years, designs generally did not include adequate provision for this action. Ties were frequently omitted or were inadequate at the top and cracks occurred under load concentrations, such as that due to a girder or beam. In many instances anchor bolts for girder seats were frequently inadequate and thus permitted failure to occur.

Where buildings have been constructed with exterior masonry walls and wood roof or floor framing, failures from seismic loads have occurred occasionally in connections between the masonry and wood framing due to lack of wall ties or other positive anchorage. Current building regulations provide special detailed requirements to protect against such failures.

14.7.2 PRE-MODIFICATION VERIFICATION OF MATERIALS

In evaluating the existing conditions of masonry structures, the framing and masonry construction systems must be identified and the types of construction materials determined since the type of failure depends on such variations. Also the structure should be examined for signs of aging and deterioration. Deterioration may have affected all portions of older masonry structures. Some examples are given below.

- Cracking due to restraint of thermal expansion or contraction occurs most frequently when sections have been reduced, as at windows or doors.

- Cementitious materials shrink upon drying and if elements are restrained they may crack. The effect is similar to that of shortening due to thermal contraction. Occasionally, shrinkage does not cause visible cracks, but may cause stresses that will influence installation details.

Although some testing may already have been done in a preliminary investigation, it may be necessary to obtain additional data in order to formulate a program of strengthening and/or repair. Specific tests for accurately determining vertical load carrying capacity, in-plane and traverse shear capacities, and bending capacity are necessary. Typical test procedures for evaluating material properties and structural capacity are as follows:

(A) NON-DESTRUCTIVE TESTS. Electromagnetic test equipment is used primarily for locating reinforcement. Various types of equipment are available, varying from the carpenter's "stud finder" which may be used for locating ladder joint reinforcing near the surface to more sophisticated detectors for locating steel bars in reinforced concrete. Such equipment would identify the location and direction of bars, depth below surface, and size of reinforcement.

Sonic tests may be useful in assessing extent of cracking in mortar joints and some percussion tests may be useful in evaluating relative variations in strength or stiffness of masonry assemblages. However, data on use of such equipment on masonry construction are not available because these techniques have not been widely used.

(B) STRENGTH TESTS. Strength tests can be conducted on various types of samples.

1. Coring. Although coring is a simple procedure, interpretation of strength tests on core specimens is difficult and the information obtained from such tests is of limited value. However, the visual observations of the masonry and mortar in the interior of the wall as revealed by coring is useful. The principal limitations of coring and core tests are:

- Core reveals local conditions which may not be representative.
- Cores are often damaged in the process of drilling and test results do not represent properties of material in place.
- Axial compression tests on cores do not represent in-plane compression strength of masonry, as the specimen axis is transverse to the wall.
- Shear-off or sliding resistance tests on cores do not reflect the shear conditions in the wall as a unit.

If good cylindrical cores can be obtained by proper drilling technique, some measure of "splitting" strength may be obtained, similar to splitting strength of concrete cylinders. However, data on validity of such tests are not available, as this type of testing has not been widely used.

2. Prisms. The cutting of prisms will give a more correct and appropriate measure of masonry strength than will cores. The size of the prism specimens should conform to the provisions of Chapter 12A dealing with determination of masonry strength in new construction. For special shear tests other prisms or small wall panel specimens may be used. When cutting of prisms is carefully handled, test results correlate well with vertical compression testing for new construction. Dependable values of shear for masonry, particularly when old lime mortar has been used, are difficult to obtain and the usual fractions of compression strength can not be used for shear values.

3. Shear Testing. For assessment of in-plane shear value, it is recommended that prisms or small wall panels be tested on the diagonal. Results of such testing have not been verified conclusively, but it appears that they give a better indication of shear resistance than extrapolation from compression test results.

4. Load Testing. Load tests may be used to determine that an element may have capacity with adequate factor of safety to resist the design loads. However, lateral loads are difficult to apply and consequently have not been used as frequently as for determining capacity under gravity loading.

5. Modulus of Elasticity. The stiffness of masonry influences its dynamic response. Both compression modulus and shear modulus should be determined by tests using compression and shear test procedures mentioned above. Estimates of the moduli values based on compression strength used in current or past regulatory provisions for new construction can lead to unrealistic values.

(C) VERIFICATION OF FIELD DIMENSIONS AND CONDITIONS. The adequacy of the preliminary investigation must be verified and additional data necessary for final design must be obtained. This generally requires the exposure of additional areas and conditions for verification of construction details. Any discrepancy from the design drawings observed upon exposure of the structure must be noted in the investigation report.

14.7.3 REPAIR AND STRENGTHENING

The problems of rehabilitating existing buildings differ in some respects from the initial design of a building. In existing buildings, size, type, wall locations, material, and orientation are fixed, reducing the freedom of design inherent with new construction.

Older structures show effects of element deterioration, chemical reactions, stresses induced by long-time foundation settlement or beam end rotation, and impact as well as the effects of seismic forces. Some design refinement is required to account for these factors, and thorough investigations of actual performance of elements in existing structures may be necessary.

Detailed economic evaluations must also precede formulation of any repair or strengthening program. In certain cases it may be more economical to replace rather than repair damaged elements. Effectiveness of repair or strengthening depends on selection of appropriate techniques, some of which are discussed below.

Shotcrete or gunite is a concrete mix pneumatically applied to a solid surface. Design of the appropriate mix, selection of correct process, and skilled workmanship are essential for successful use of shotcrete. Restrictions on clearance, thickness, direction of application, lightweight vs. heavyweight aggregate material, etc., must be taken into account.

Prestressing may be used, placing masonry under continuous compression and thus minimizing tension failures. Vertical rods are inserted into vertical drilled cores, threaded into foundation anchorages, and stressed to an appreciable level, preventing tension due to lateral load. Brickwork is especially suitable for prestressing because the prestress is not relieved by creep or shrinkage as it is in concrete.

Plaster has been effective in bonding reinforcing mesh to masonry surfaces. The process is effective because the reinforcement is near the outer fibers where it will resist bending stress most effectively.

Surface bonding is a recent technique wherein glass-fiber reinforced cement plaster is bounded to masonry surfaces. A new ASTM standard has been initiated and new design data are being developed for this technique.

Epoxies which are high-strength adhesives have proven to be highly effective in repairing masonry elements. Epoxy burns or deteriorates at about 400°F and therefore can not be relied upon to survive exposure to fire.

14.7.3 Cont.

Some newer foaming adhesives are advantageous, especially for filling large and irregular cavities. However, in using these materials their performance in masonry units exposed to fire should be considered.

Cement grout has been satisfactorily intruded into cracks and interstices. Repair by dry-packing with nonshrinking portland cement aggregate mix after removal of damaged masonry material is effective and economical. Braces and stiffening elements of metal or other material can be attached to masonry components using bolts or other anchorages set with mortar or polymeric adhesives in drilled holes.

Procedures used to design and construct older masonry structures were not precise; having been developed as rule-of-thumb rather than as engineered provisions. Therefore, past regulatory design provisions have been neither refined nor always correct. Although current provisions are sound for average new masonry, they may be inaccurate for older structures, potentially causing serious problems in developing programs for rehabilitation.

14.7.4 VERIFICATION OF MODIFICATION

Thorough inspection of repair and strengthening work is at least as important as it is for new construction. As conditions are disclosed, the engineer must be kept advised of any deviations and resulting changes must be noted. A complete work record or job diary should be maintained. The condition of portions of structures exposed, prepared, and repaired must be recorded. Photographs should be taken to record all conditions.

Sec. 14.8 FOUNDATIONS

Foundations of existing buildings may be damaged by earthquake or wind forces, overloading under gravity loads, poor soil conditions, landslide, or excavation on adjoining property. Where a building is founded on loose or not-very-dense soils, ground shaking may consolidate the soil, causing the supported structure to settle. Liquefaction may occur when certain types of soils containing excessive moisture are severely shaken.

Overturning effects induced by earthquake forces create positive and negative pressure on footings. Such pressure may vary linearly under a shear wall or may be concentrated at or near the ends of a wall depending on the configuration of the footing system. Where moment-resisting frames are used, columns may be subjected to additional direct stress, either tension or compression, which may cause differential settlement.

14.8.1 PRE-MODIFICATION VERIFICATION

Damage to foundations above grade, such as cracks in grade beams and basement walls, may frequently be assessed by visual observation. From the crack pattern, the direction of settlement can usually be determined. From the size of cracks, the degree of settlement can be estimated. Surveying instruments may be used to refine visual assessments. Field observation may show the structure to be adjacent to a hillside where the slope or soil stratification could contribute to sliding.

The best method of determining causes of settlement and resulting damage is to review the original soils investigation report for the site, if such a report exists. Where such a report does not exist, a soils investigation should be made. Exploration below grade can often be made to determine the condition of footings and piles. In the case of pile footings, if significant lateral movement has occurred the piles may have been damaged due to excessive bending. This can be verified by uncovering a length of pile. However, removal of earth from the pile surface will reduce the load-bearing capacity of friction piles. In some cases, damage may be so great or soil properties so poor that repair or strengthening is not economically feasible.

14.8 Cont.

14.8.2 REPAIR AND STRENGTHENING

Cracks in concrete or masonry foundation elements may be repaired as described in Sec. 14.4 and 14.7. Footing elements may be strengthened by adding cross-sectional area reinforcement as described in these sections. Where epoxy injection is used to repair cracks in exterior basement walls, the injection may be limited to one side. In such cases it may be difficult to obtain full penetration, and special care must be exercised in determining adequacy of such repair.

Correcting settlement distress involves both preventing future settlement and releveling. The method used to correct settlement distress will generally depend on the supporting soil characteristics and a thorough soils investigation should therefore be made and evaluated prior to selecting a method. Some common methods for correcting settlement are described and discussed below.

The size of spread footings shown by analysis not to conform to relevant code provisions may be increased by underpinning. Underpinning an existing footing in strips may eliminate the necessity for shoring. Where underpinning is used, the footing must be evaluated for capacity to resist shear and bending moment as anticipated by design criteria mandated by the code. If footings are still inadequate, they may be removed and replaced with larger footings.

Where space permits, new footings may be placed on each side of an inadequate existing footing, and a beam or beams run between the new and existing footings so as to carry the existing load. Similarly, where space permits caissons or piles may be used in lieu of spread footings and placed on each side of the existing footing.

In underpinning operations, provision should be made for jacking up the settled footing; this requires space between underpinning elements and the existing footing. After the settled component has been leveled, this space can be filled with concrete and drypacked. Drypacked grout should have low shrinkage characteristics.

Pile footings that have settled due to earthquake forces are usually more difficult to relevel and strengthen than spread footings. Soil stabilization, discussed below, rather than underpinning should be considered here. Where possible, such procedures obviate the need for additional, difficult-to-install piles. Where calculations show that multiple pile footings do not meet appropriate code criteria, it may be necessary to remove the existing pile cap, place additional piles, and provide a new pile cap. The existing column or other load-bearing vertical element must be cut loose and jacked back to its required position. Such vertical components must be temporarily supported before being cut loose.

Space for adding new piles must be available, including vertical clearance for a pile driver if driven piles are used or space for a drill rig where drilled and cast-in place concrete piles are used.

Some methods of soil stabilization and compaction, such as vibration, preloading, and blasting, increase surface settlement of the area and therefore cannot be used to repair foundations. Other methods, such as pressure grouting or intrusion grouting with cement grout or chemical grouting, do not settle the ground surface and are frequently used to stabilize soils under existing buildings in that the bearing capacity of the soil is increased. Pressure grouting may also be used to raise settled footings or floor slabs.

Soil stabilization should never be used unless a thorough investigation of underlying soils has been made. Selection of a soil stabilization technique is a function of specific soil characteristics, such as size of granules, moisture content, and soil chemistry. Some soils are not adaptable to any type of soil stabilization technique. Soils stabilization is both a science and an art, requiring the joint effort of a soils engineer and an experienced contractor with proper equipment. The primary purpose of soil stabilization in the repair of existing structures is to densify the soils supporting the building footings and thus to increase the load-bearing capacity of soil where settlement has occurred and calculations indicate that footing size or original soil capacity does not conform to appropriate code provisions.

Compaction grouting can be used to densify soil, to increase load-bearing capacity, and/or to raise or level footings and slabs. Soil-cement grouts or chemical grouts may be used. When soil cement grout is used, the method is sometimes called "mudjacking". Where a large volume of soil must be stabilized, grouting is expensive. With proper equipment, pressure grouting can be accomplished to depths of 50 feet or more. In one case, a building was founded on bell-bottom caissons about 50 feet in depth, below which the fill was poorly compacted. Satisfactory results were obtained by pressure injecting cement grout into the soils beneath the belled footing.

Chemical grouts are normally applied into activated or partially saturated formations. However, dry granular, or fractured formations may be grouted, usually when the purpose is to increase the strength of soils rather than to shut off water. A complete discussion of chemical grouting is beyond the scope of this document. Soil stabilization should only be used when a complete soils report is available and the soil chemistry is such that chemical grouting is appropriate. A qualified soils engineer and grouting contractor experienced in this field should be employed for chemical grouting. In some cases, investigation may indicate that the site itself is unsafe or that it is not economically feasible to stabilize the soil. In such cases, abandonment and demolition of the building may be necessary.

Sec. 14.9 NONSTRUCTURAL COMPONENTS

Guidelines for evaluating nonstructural components in existing buildings and for determining necessary levels of seismic resistance are described in Chapter 13. The seismic design requirements for nonstructural components in new buildings are given in Chapter 8. Some items mentioned in Chapter 8 are not covered in this Section, but methods of correcting the most frequently found hazards are reviewed.

Since structural quality and anchorage of nonstructural components in existing buildings vary widely, it is not feasible to describe in detail methods for repairing or strengthening all such components, especially for mechanical and electrical equipment where there are many different types and models. Such equipment is frequently designed and manufactured without provision for seismic resistance. Sound engineering judgement is essential for determining the best method of repairing or strengthening such components. Economic feasibility should be weighed against seismic risk in repairing nonstructural components as should be done for buildings as a whole.

Nonstructural components such as nonbearing corridor walls and stair enclosures are particularly critical, since their failure may make it difficult or impossible to exit in case of an emergency. Similarly, it is critical to consider the effect of the proximity of nonstructural components to essential emergency equipment if the failure of such components could cause damage to the equipment or limit accessibility to it.

Where it is more expensive to repair or strengthen nonstructural components than to replace them with components or systems conforming to specified design criteria, replacement should be considered.

14.9.1 PRE-MODIFICATION VERIFICATION

Once the method of determining the need for strengthening has been determined, components should be re-evaluated with due consideration for life safety and possibility of replacement prior to the time that a decision to repair or strengthen is made.

Parapets, ornamental exterior appendages, marquees and canopies, and fire escapes could constitute a hazard to life if dislodged during an earthquake, especially where such components are located above and adjacent to a public way or adjoin another building. Ceilings, light fixtures, mechanical and electrical equipment, elevators, nonbearing partitions, and stairs could also constitute a hazard to life were they to fall during an earthquake. Window glass may be a hazard where it cannot accommodate the story-to-story movement of a structure without breaking or if the glazing cannot accommodate deformations due to seismic forces. The anchorage and attachment of nonstructural components to the structural system, particularly with respect to corrosion and details of installation, must be evaluated for capability to accommodate earthquake forces and displacements as anticipated by appropriate code criteria, and must be made to conform to code requirements where deficient.

14.9.2 REPAIR AND STRENGTHENING OF NONSTRUCTURAL COMPONENTS

The repair and strengthening of nonstructural components are described in this Section.

(A) PARAPETS. In some cases unreinforced masonry parapets can be strengthened by reducing their height to a level where the parapet acting as a cantilever can resist earthquake forces induced from its own mass. Where parapets act as cantilevers, the adequacy of the walls below the anchorage level must be evaluated for capacity to withstand cantilever moment as required by the code. In cases where it is desirable to maintain a certain height, a cap beam of steel or reinforced concrete may be installed at the top of the parapet and the cap beam braced as necessary to the roof structural system. Cap beams are designed to span braces, and the attachment of such braces to the roof system must be evaluated for strength to resist earthquake forces and made to conform to code requirements where deficient.

Reinforced concrete and reinforced masonry parapets may be capable of resisting lateral forces. If calculations show that such parapets cannot resist the required earthquake forces, they may be reduced in height and re-anchored as required or strengthened so as to conform to code requirements.

Unreinforced concrete walls can be corrected as described for masonry walls in Sec. 14.7, or can be strengthened by adding reinforced shotcrete on each side of the wall where additional weight is acceptable.

Wood parapets or parapets of wood and plaster fall into two categories: (1) studs or other vertical elements extend from the story below and are, or can be, anchored to the roof and (2) vertical studs or other elements start at or near the roof line and the only resistance to cantilever moments induced by seismic forces forces is provided by the nailing of the sill plate. In this case, bracing to roof is necessary.

(B) APPENDAGES AND VENEERS. Ornamental appendages are made of a number of different materials and take almost any shape and size. Cornices or terra cotta appendages, sheet metal, concrete, stone or cast stone, statues, and gargoyles are categorized as appendages and veneers. The strength of appendages and that of their anchorage to the structural system are the important factors to be considered in a repair or strengthening program. The anchorage of appendages in older buildings may be impaired by corrosion, or the material into which the anchors have been embedded may have deteriorated or been damaged. It is difficult to evaluate the condition of anchorages unless they are exposed. Appendages not conforming to the specified requirements may be removed or, if this is not desirable, appendages and/or their anchorages may be strengthened so as to conform to code requirements.

Masonry veneers may be categorized as either adhered or anchored. Adhered veneer is attached to a structure as a whole or to some component by an adhesive cementing material. Frequently, such veneer will suffer the same distress as that suffered by the supporting component. However, in some cases deflection of the supporting component may only induce bond failure between the structural component and the veneer. This type of failure may also be caused by temperature changes, shrinkage, or deflection.

The failures in anchored veneer may be disguised even more frequently than adhered veneer. Since anchored veneer is usually thicker, heavier, and stiffer than adhered veneer, it can normally tolerate more lateral force; anchors and ties may permit considerable differential motion between the structural face and the veneer. Cracks in the surface component may be quite different from those in the covered supporting component and thus the evaluation of damage is complicated. Where the number of damaged veneer elements is small, it is usually best to replace them and ensure that the anchorage or adhesive conforms to code requirements. The replacement of ornamental patterned veneers of webbed terra cotta or cast stone is difficult and expensive unless original forms are available.

(C) CEILINGS. Plaster ceilings may be on wood or metal lath, and the lath may be attached directly to the soffit of wood joists or may be suspended with wood or wire hangers. Where ceilings are attached directly to the soffit of wood structural members, such as joists, nailing must comply with code requirements. In older buildings, nailing may have loosened and new nails or screws with proper pull-out values should be installed.

Most modern ceilings are supported by horizontal metal frames in more or less standard modules. Vertical hangers are attached to these frames at intervals, depending on the strength of the frame, and to the supporting floor or roof structure above. These hangers usually have no significant compressive strength. Tension diagonal bracing can be installed to transmit the lateral force from the ceiling to the floor or roof system. Under such conditions, framing members between the tension diagonals will be in compression and must be evaluated for code compliance. Vertical accelerations of supporting components may also produce compression in some bracing members.

In existing buildings in areas of high seismicity, horizontal force factors compatible with geographic location and occupancy risk as set forth in Chapter 13 should be considered in strengthening suspended ceilings.

(D) LIGHT FIXTURES. Light fixtures may generally be classified as recessed, surface-mounted, or pendant-hung. Light fixture support systems must be evaluated for their capacity to resist lateral force and strengthened where deficient as required in Chapter 13.

(E) MARQUEES AND CANOPIES. The anchorage of marquees and canopies is usually critical, particularly in the case of older buildings where corrosion or anchorage to impaired materials may have rendered such components unsafe. Such anchorages should be exposed to determine their condition. If they do not conform to code requirements, they should be replaced or strengthened, possibly requiring that additional structural members be installed inside the building if calculations show that existing anchorage to the walls or structural system cannot resist the prescribed forces.

(F) FIRE ESCAPES. Fire escapes in older buildings may not have been recently examined. In the event of an earthquake, fire escapes may have to function as exits if stairs are blocked. Corroded anchorages or those embedded in damaged or deteriorated material may not provide lateral resistance specified by the code, and if so must be strengthened to comply. Fire escapes should be load tested where practical. If such a procedure is impractical on a large scale, a detailed visual inspection should be made to identify defects requiring correction.

(G) NONBEARING PARTITIONS. Nonbearing masonry partitions may be repaired or strengthened as described in Sec. 14.7. The methods of repair and strengthening of nonbearing, nonstructural concrete or wood walls are similar to those described in Sec. 14.4 and 14.6.

Earthquake-induced damage to metal stud walls with plaster or dry wall partitions usually occurs in the anchorage to the supporting structure and in cracking of brittle surfaces. Where anchorage does not meet the recommended requirements, it should be strengthened with additional screws, bolts, etc. Cracks in plaster or other wall covering may be repaired using current techniques.

Where structural analysis indicates that it is not desirable for a partition to resist in-plane earthquake forces, it should be isolated from such forces, but must be capable of resisting earthquake forces normal to the wall. The width of separation at the ends of the wall should be sufficient to accommodate the anticipated story drift. Partition isolation may violate fire separation requirements. If so, appropriate remedies must be developed to comply with relevant code provisions.

(H) MECHANICAL AND ELECTRICAL EQUIPMENT. Mechanical or electrical equipment may be supported by structural frames. Where such equipment is necessary to life safety or is located where it would be hazardous to life if dropped or overturned, the stability of such equipment must be investigated. The required level of earthquake resistance for equipment in existing buildings is given in Chapter 8. Where these criteria are not met, the equipment and the lateral and vertical support systems should be strengthened. Often a detailed analysis of portions of the equipment and support system anchorage must be made.

(I) STAIRS. Stairs provide emergency exits in the event of an earthquake or other disaster. Stairs may be constructed of wood, steel, or concrete, and, although those in existing buildings may have withstood the test of time for vertical loads, their safety under lateral forces must be determined. In an earthquake a stairway may act as a diagonal brace between floors and therefore must be capable of accommodating story-to-story drift without damage.

Stair enclosure partitions or walls may fall into stairwells and block the stairs when an earthquake occurs. Stair enclosure partitions or walls found by analysis not to conform to code requirements may be repaired or strengthened as described in Sec. 14.4 through 14.7. The repair and strengthening of nonbearing partitions is discussed in Sec. 14.9.3(G).

(J) ELEVATORS. Criteria for the seismic design of elevators, elevator equipment, and elevator guides are provided in Chapter 8. During an earthquake, the guides of elevator systems may fail or their anchorage to the structural components may be inadequate. Elevator systems may be anchored to enclosure walls that do not have the capacity to resist lateral forces or deformations prescribed by the code. Such enclosure walls may be repaired or strengthened as described in Sec. 14.9.3(K) for stairs. Elevator guide anchorages should be evaluated for conformance with the level of seismic resistance required and strengthened as necessary. Guides for cabs and counterweights should be evaluated for required strength and ability to accommodate differential displacements. Anchorage of motor generator equipment and controllers to their support beams should be repaired or strengthened as described in Sec. 14.9.3(H). An engineering analysis will usually be necessary for each condition and component. Corrosion and deterioration should also be considered.

During earthquakes in Japan, California, and Alaska elevators were severely damaged in a variety of ways:

- Motor generator sets were thrown off un-anchored isolation mounts.
- Control panels toppled over where not anchored to structural frames.
- Control relays were damaged when unlatched and hinged panels thrown open.
- Counterweight guide systems were damaged with counterweights derailed or allowed to swing, possibly impacting cars.

Hydraulic elevators which are usually found in low buildings have suffered little earthquake damage except where ground rupture has overstressed the hydraulic cylinders.

(K) CHIMNEYS. Chimneys at a building perimeter are normally supported laterally by ties to the supporting structure to prevent their falling outward. Interior chimneys sometimes are enclosed by the structural system without being connected to it.

Ties of chimneys should be evaluated where the supporting structure is a relatively flexible frame building and the chimney is a stiff masonry element. Chimneys often fail due to bending above the roof or point of lateral support during an earthquake. Older chimneys may deform laterally when the lime-mortar bed joints in unreinforced masonry fail. Such failure can be detected by sighting up the chimney and noting any offsets.

Chimney ties in multistory buildings frequently fail with the chimney falling away from the structure during an earthquake. Anchorage of ties should be evaluated and made to conform to code requirements when necessary. Increasing the size of such anchors may not be advisable because heavier ties are more difficult to attach and the critical point needing strengthening may be bolts, nails, screws, etc., which provide the anchorage attachment.

Chimneys may be repaired by replacing damaged portions or by providing new or additional ties. Strengthening of unreinforced masonry chimneys may also involve replacement with reinforced masonry. Where space permits, it may be possible to apply reinforced shotcrete to chimney walls as described in Sec. 14.7, by removing one wythe of masonry prior to applying shotcrete, or by adding shotcrete where additional weight would not be detrimental.

For reinforced concrete chimneys repairs may be made to the chimney walls as described in Sec. 14.4. Reinforced concrete chimneys may be strengthened by adding reinforced shotcrete to the walls or by adding additional ties so that the distance between lateral supports is reduced.

(L) WATER STORAGE TANKS. Water storage tanks supported on buildings must be evaluated for vertical and lateral load-bearing capacity and strengthened, where necessary, to conform to code requirements. Steel framing and connections should be checked for deterioration. If necessary such tanks should be removed and replaced.

TABLE 14-1

CHECKLIST FOR EXISTING BUILDING DATA FILE

A. <u>Original Construction</u>

1. Date of completion

2. Plans and specifications availability

   a. Available

      Field check for conformance

   b. Not available

      Field survey and measurements required
      Exploratory probing may be necessary

3. Basic code prevailing and year of promulgation

   a. Established by what governmental agency having jurisdiction

      Uniform Building Code (UBC) - International Conference of Building Officials (ICBO)
      BOCA Basic Code - Building Officials Conference of America, Inc.
      National Building Code - American Insurance Association
      Building Standards, Title 24, State of California
      SBCC - Southern Building Code Congress
      Applicable local codes (city or county)
      Applicable state code
      Applicable federal code (FHA, HUD, etc.)
      Other applicable code

   b. Industry standards referenced in basic code

      AISC - Specifications and codes - American Institute of Steel Construction Inc.
      ASTM - American Society for Testing Materials
      AWS - American Welding Society
      Steel Joint Institute
      AISI - American Iron and Steel Institute
      ASNDT - American Society for Nondestructive Testing
      ACI - American Concrete Institute
      PCA - Portland Cement Association
      PCI - Prestressed Concrete Institute
      NCMA - National Concrete Masonry Association
      SCPI - Structural Clay Products Institute
      BIA - Brick Institute of America
      Product and Commercial Standards - NBS, Dept. of Commerce
      NFPA - National Forest Products Association
      APA Standards - American Plywood Association
      AWPI Treating Standards - American Wood Preserver Association

(TABLE 14-1 CONTINUED)

B. <u>Review Mandated Level of Structural Sufficiency Established by Governmental Agency Having Jurisdiction</u>

1. Determine adequacy of existing structure for:

   a. Gravity loads
   b. Seismic loads; refer to Chapter 13
   c. Impact or dynamic loads

2. Determine necessary modification to achieve achieve mandated level of structural integrity for:

   a. Gravity loads
   b. Seismic load resistance; refer to Chapter 13
   c. Impact or dynamic loads

C. <u>Develop Design Document to Implement Hazard Mitigation Criteria Established by Regulatory Agency</u>

Extract pertinent procedural information from the Chapter 14 text on the respective construction materials which will assist in establishing safe design review physical properties.

Make note of any special situations which may suggest a later need for a subjective engineering evaluation after all the facts are known.

14 Cont.

TABLE 14-2

GUIDE FOR USE OF NONDESTRUCTIVE EXAMINATION OF WELDS

DISCONTINUITY	LOCATION			GROOVE JOINT TYPE																FILLET WELD TYPE											
				Butt				Corner				Tee				Lap				Corner				Tee				Lap			
Type	W	H	B	U	R	M	L	U	R	M	L	U	R	M	L	U	R	M	L	U	R	M	L	U	R	M	L	U	R	M	L
1. POROSITY																															
a. Uniform Scattered	X	O	O	c	a	c	b	c	d	c	b	c	d	c	b	b	b	c	b	d	d	c	b	d	d	c	b	d	c	c	b
b. Cluster	X	O	O	c	a	b	b	c	d	b	b	c	d	b	b	b	b	b	b	d	d	b	d	d	d	b	b	d	c	b	b
c. Linear	X	O	O	c	a	b	b	c	d	b	b	c	d	b	b	b	b	b	b	d	d	b	b	d	d	b	b	d	c	b	b
d. Piping	X	O	O	c	a	b	b	c	d	b	b	c	d	b	b	b	b	b	b	d	d	b	b	d	d	b	b	d	c	b	b
2. INCLUSIONS																															
a. Slag (Nonmetallic)	X	O	O	b	a	c	d	b	d	c	b	b	d	c	b	b	a	c	b	d	d	c	b	d	d	c	b	d	d	c	b
b. Tungsten (Metallic)	X	O	O	c	a	c	d	b	d	c	d	c	d	c	d	c	a	c	d	d	d	c	d	c	d	c	d	c	d	c	d
3. INCOMPLETE FUSION	X	O	O	a	a	b	b	a	d	b	b	a	d	b	b	a	a	b	b	d	d	b	b	c	d	b	b	c	d	b	b
4. INCOMPLETE JOINT PENETRATION																															
a. Inadequate Joint Penetration	X	O	O	a	d	d	d	a	d	d	d	a	d	d	d	a	d	d	d	d	d	d	d	c	d	d	d	c	d	d	d
5. UNDERCUT	O	O	X	a	a	c	c	a	d	c	b	a	d	c	c	a	a	c	c	d	d	c	c	d	d	c	c	d	d	c	c
6. UNDERFILL	X	O	O	a	a	d	d	a	d	d	d	a	d	d	d	a	a	d	d	d	d	d	d	d	d	d	d	d	d	d	d
7. OVERLAP	X	O	O	a	b	b	b	a	d	b	b	a	d	b	b	a	b	b	b	d	d	b	b	d	d	b	b	d	d	b	b
8. LAMINATION	O	O	O	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a
9. DELAMINATION	O	O	O	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a	a	d	a	a
10. SEAMS AND LAPS	O	O	O	b	b	a	a	b	d	a	a	a	d	a	a	a	b	a	a	b	b	a	a	b	d	a	a	b	d	a	a
11. LAMELLAR TEARS	O	O	O	a	d	d	d	a	d	c	c	a	d	c	c	a	d	c	c	b	c	c	c	b	d	c	c	b	d	c	c
12. CRACKS																															
a. Longitudinal	X	X	O	a	b	a	a	a	d	a	a	a	d	a	a	a	b	a	a	c	c	a	a	c	c	a	a	c	c	a	a
b. Transverse	X	X	X	a	b	a	a	a	d	a	a	a	d	a	a	a	b	a	a	c	c	a	a	c	c	a	a	c	c	a	a
c. Crater	X	O	O	a	a	a	a	a	d	a	a	a	d	a	a	a	a	a	a	c	c	a	a	c	c	a	a	c	c	a	a
d. Throat	X	O	O	a	b	a	a	a	d	a	a	a	d	a	a	a	b	a	a	c	c	a	a	c	c	a	a	c	c	a	a
e. Toe	O	X	O	a	b	a	a	a	d	a	a	a	d	a	a	a	b	a	a	c	d	a	a	c	d	a	a	c	d	a	a
f. Root	X	O	O	a	b	a	d	a	d	c	d	a	d	c	d	a	b	c	d	c	d	c	d	c	d	c	d	c	d	c	d
g. UNDERBEAD AND HAZ	O	X	X	a	b	c	d	a	d	c	d	a	d	c	d	a	b	c	d	c	c	c	d	c	c	c	d	c	c	c	d
h. Fissures	X	O	O	a	b	d	d	a	d	d	d	a	d	d	d	a	b	d	d	c	d	d	d	c	d	d	d	c	d	d	d

Legend:

W = Weld	U = Ultrasonic	a = Good
H = Heat Affected Zone	R = Radiographic	b = Limited
B = Base Material	M = Magnetic Particle	c = Poor
X = Applicable	L = Liquid Penetrant	d = Not usually applicable
O = Nonapplicable		

TABLE 14-3

TYPICAL STRENGTH AND STIFFNESS CHARACTERISTICS
OF SOME REPAIR MATERIALS
(pounds per square inch)

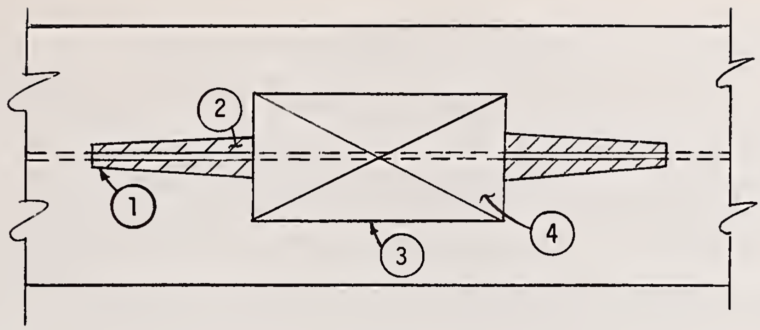
Material	3-Day Compressive Strength	28-Day Compressive Strength	3-Day Tensile Strength	28-Day Tensile Strength	2-Day Modulus of Elasticity	28-Day Modulus of Elasticity
Epoxy - Neat	10,400	12,000		4,900*	260,000	490,000
Epoxy - Mortar	8,400	9,900	740	920	$2x10^6$	$2x10^6$
Gypsum Cement Mortar	4,800	7,200	430	570	$3.2x10^6$	$4.2x10^6$
Quick Setting Cement Concrete Mortar	5,000	7,900	420	610	$3.6x10^6$	$4.2x10^6$

*Based on 14 days.

TABLE 14-4
PERCENT INCREASE OR DECREASE IN RECOMMENDED DESIGN VALUES
FOR EACH 1°F DECREASE OR INCREASE IN TEMPERATURE

Property	Moisture Content	Cooling, below 68° F (-300° F min.)	Heating, above 68° F (150°F max.)
Modulus of Elasticity	0%	+0.04%	-0.04%
	12%	+0.14%	-0.19%
Other Properties	0%	+0.17%	-0.17%
	12%	+0.32%	-0.49%

14 Cont.

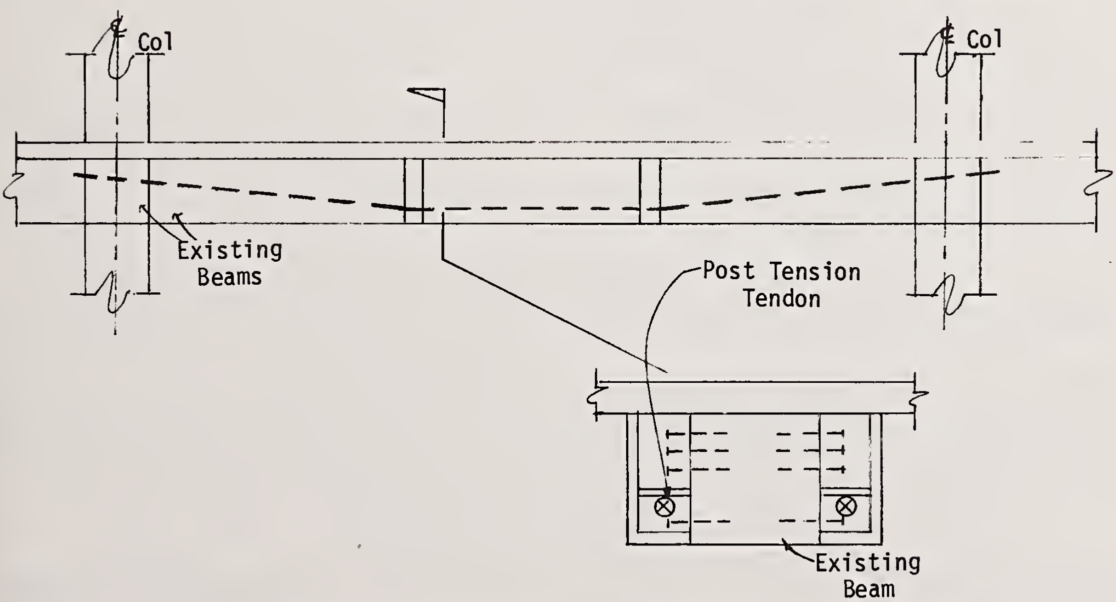


Procedure:

1. Cut slots in concrete exposing tendon for length required to develop force and ultimate strength.
2. Grout tendon in slot using nonshrink concrete, expoxy grout, etc.
3. Cut required hole.
4. When bonding material has developed strength, gradually reduce the force in the tendon remaining in the hole by heating it with a cutting torch. Cut tendon and remove.

HOLE CUT THROUGH CONCRETE SECTION INTERRUPTING UNBONDED TENDON
(stress in remainder of tendon must be maintained)

FIGURE 14-1



INCREASE CAPACITY OF REINFORCED CONCRETE BEAM BY POST TENSIONING

FIGURE 14-2

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CHAPTER 15

GUIDELINES FOR EMERGENCY POST-EARTHQUAKE INSPECTION AND EVALUATION OF EARTHQUAKE DAMAGE IN BUILDINGS

Sec. 15.1 INTRODUCTION

This Chapter presents guidelines for emergency post-earthquake inspection and evaluation of earthquake damage in buildings, a statement of the problem, and commentary.

In the event of a disaster such as a major earthquake, building officials may be faced with chaotic and confusing circumstances due to the lack of warning, delays in locating the damage areas, possible lack of communications, restricted mobility, requests for damage inspection and first assessments of the dollar damage, etc. As a consequence it is anticipated that normal building department functions will cease and personnel will be allocated to administer the emergency situation. (SEAOC, 1971)

In his address to the Japanese-American Conference of Mayors and Chamber of Commerce Representatives at Kyoto, Japan, October 23, 1971, the Honorable Edwin W. Wade, Mayor of Long Beach, California, stated:

> "Whereas the federal government now provides a considerable amount of assistance, it remains for the city to withstand the first massive onslaught of the disaster. The city stands alone during those first frightful hours, or perhaps days, and it is during this initial period of time when the good building codes, the good communications, the good decision making, and the good planning pays off in terms of lives and property saved. What is done or what is not done during this early period more than anything else will determine how well the public trust has been preserved." (Office of Emergency Preparedness, 1972)

The validity of the statement by Mayor Wade was dramatically demonstrated by the February 9, 1971 San Fernando earthquake. Most building departments were not staffed to handle this type of emergency disaster situation and were swamped in attempting to respond to the immediate problems mentioned above. As a consequence, it makes abundant good sense for local jurisdictions to develop mutual-aid plans with other governmental jurisdictions, and to enlist the aid of private sector engineers to assist building departments in the emergency assessment and evaluation of damage to buildings.

When a major disaster strikes, the sequence of events that occurs at the government level is as follows (Federal Disaster Assistance Handbook for Government Officials, undated):

(A) The local authority quickly assesses the magnitude of damage and decides whether or not help is needed from the state government.

(B) In the event that help is needed from the state government, the mayor of the city will request aid from the state, and the state will give all possible assistance.

(C) If the magnitude of the disaster is considered beyond the state's capability, the governor of the state will request either a presidential declaration of a major disaster or a determination of an emergency through the Federal Disaster Assistance Administration (FDAA) in order to implement the provisions of PL 93-288, the Disaster Relief Act. (Federal Disaster Assistance Program-Eligibility Handbook Under Public Law 93-288, 1974)

(D) The regional office of the FDAA then surveys the damage and makes a recommendation to the FDAA's regional director who evaluates the request and recommends action to the President.

(E) The President determines whether the request is warranted and, if it is, declares a "major disaster" and may allocate funds. The President informs the governor and directs the FDAA to reallocate funds and direct and coordinate disaster relief.

The essential information needed by local jurisdictions for state and federal jurisdictions is an estimate of dollar damage and the distribution and severity of damage. Organization of resources and activities for the pre-earthquake preparation period and for the emergency post-earthquake period is shown in Exhibit 15-1.

Sec. 15.2 OBJECTIVE AND SCOPE

The objective of this Chapter is to present procedures for examining and reporting on damage to buildings immediately following a major earthquake in a highly urbanized community. Although the recommendations and procedures outlined will be most seriously considered in seismically active regions of the country, it should be remembered that some relatively seismic-inactive areas have experienced quite large earthquakes. The possibility of their occurrence should not be ruled out in any area. In addition, the organization, training, and procedures will be valuable in other disaster situations such as those caused by tornados and floods.

It is important to recognize that the ability to effectively accomplish the objectives of this Chapter is predicated on the assumption that there is available to the affected area a large group of motivated people who are enthusiastic and qualified for the roles they are intended to fulfill. It is proposed that state-wide plans be developed to recruit, train, mobilize, equip, and deploy professional engineers from the private sector as deputies of the building official to inspect damaged buildings. The Structural Engineers Association of Northern California conducted a study of such a plan using the questionnaire shown in Exhibit 15-2 and interest was expressed in volunteering for such duty. Several difficulties became immediately apparent, however:

(A) Some form of hold-harmless protection against personal liability of engineers must be legislated. See Exhibit 15-3 for Suggested Emergency Inspection Ordinance.

(B) Some standard means of identification such as a badge or card must be issued to the engineers so they may gain access to damaged areas.

In areas where such resources are not available, a careful reassessment of the material of this Chapter will have to be made to accommodate local conditions.

The primary purpose of these procedures is to reduce the incidence of death and injury to occupants of buildings which have been weakened or placed in jeopardy by seismic activity. The likelihood of such buildings being repeatedly subjected to earthquake forces from aftershocks during the first few days and weeks following the initial shock is great--thus there is an urgent need for early post-earthquake inspections. The secondary purpose of these procedures is to obtain an approximate idea of the magnitude of the disaster in terms of dollar value of damage and numbers of buildings affected. These data are needed by all levels of government almost immediately following any disaster in order that proper planning for aid to the area may begin. One of the greatest challenges a building department faces in a major earthquake is to furnish information and statistics to other governmental agencies such as the Small Business Administration, Department of Housing and Urban Development, state Offices of Emergency Preparedness, Federal Office of Emergency Preparedness, Veterans Administration, Corp of Engineers, Red Cross, local housing authorities, private lending institutions, and the news media.

Most of the information can be obtained during initial emergency inspections and the data collected transferred to the Emergency Earthquake Damage Inspection Forms (Exhibit 15-4). This Chapter proposes that the data thus collected be transferred to computer punch cards, and stored in a computer. Once the data is punched and stored, it can be sorted and manipulated in any manner for the statistical data desired. The forms proposed in this Chapter have been designed with this capability in mind. They evolved from other forms proposed in previous years, such as those of the Earthquake Engineering Research Institute, but are designed to fulfill the specific practical demands of an emergency situation. Most earlier forms were developed primarily as research aids, but the forms in this Chapter are directed to the problems of formulating an evaluation of the safety of the structure under emergency "high pressure" conditions. The categories to be filled out are intended to force the inspector to think in a systematic manner and to insure that only what is believed to be essential data are gathered. The forms were used following the Oroville earthquake by people unfamiliar with them and produced the required information.

In preparing this Chapter, emphasis has been placed on the experience of governmental officials charged with disaster relief, and particularly those whose responsibility includes the determination of use of buildings to house disaster victims. A list of reference publications and a bibliography from which further information can be obtained are included at the end of this Chapter. There seems to be general agreement among authorities that existing governmental agencies within a damaged area should be primarily responsible for carrying out the required functions--in this case, assessment of building damage. "The key to effective organization for response to natural disasters or any other emergency is simply to accelerate and reinforce existing, practical governmental functions. An emergency is not the time to introduce a new and unfamiliar apparatus for coordination The public looks to established political authorities to act quickly and effectively in an emergency." (Office of Emergency Preparedness, 1972). Accordingly, this Chapter proposes that the agency which regulates building inspection and safety within the city, county, or other concerned local jurisdiction would be responsible for conducting the damage surveys (see Exhibit 15-3). Recognizing that such an agency would probably not be adequately staffed for the task, mutual aid from neighboring communities would be required. Certain types of personnel would probably have to be imported from remote areas in the case of a very large disaster. The state and federal governments are assumed to support the local agency in this particular function.

In general, the federal government acts as a fact-finding body with respect to the extent and dollar volume of damage experienced. Their activities generally start some time after the period considered in this Chapter and are directed more to the administration and distribution of federally authorized disaster relief funds. However, due consideration should be given in any disaster plan to interaction with the U.S. Army Corps of Engineers and regular U.S. Army units. These units should be available on very short notice for guard duty and furnishing transportation and means of communication.

The building department, as part of its disaster plan, must have methods available to contact the volunteer engineers, means to transport them to emergency operation centers, stockpiles of emergency Earthquake Damage Inspection Forms available, stockpiles of equipment for engineers' use, and means to transport engineers to damage areas (SEAOC, 1971). The make-up of inspection teams and the type of personnel needed will depend on the classes of structures to be investigated. Dwelling houses, one- and two-story apartment buildings, and commercial structures such as would be encountered in the average neighborhood shopping center could be examined by architects, building inspectors, and construction foremen. Larger structures should be examined by professional engineers experienced in structural design of buildings. Mr. Alfred Goldberg, former Superintendent, Bureau of Building Inspection, City and County of San Francisco advises as follows:

> "There has to be emphasis placed upon establishing an extremely high priority for inspection immediately after the quake of the critical facilities including hospitals, police and fire stations, etc. In addition, standby emergency

facilities such as public assembly open space facilities (gyms, etc.) should be checked for emergency housing or hospital use. Unless this is done as the first priority, the use of some facilities will continue unknowingly with danger of aftershocks to the occupants. The reliance for recruitment of public agency personnel is unrealistic. While there are some large governmental agencies such as the City of Los Angeles that have many engineers, most have few and these will have to staff office activities so as to provide public information and coordination of the disaster effort. The inspection staff is usually not knowledgeable in any but the simpler wood frame type structures, and architects, building inspectors, and construction foremen could inspect, and report damage and hazard to occupancy. However, it will take engineers experienced in structures, and preferably with post-seismic evaluations, to do the bulk of the job on the major structures. This must be emphasized."

Help from the private sector will be needed regardless of the type of area affected by a disaster, whether it be a sprawling community covering many square miles with predominantly small buildings, or a densely built area with many major structures. In this regard, the experience of the officials involved with inspection of buildings following the San Fernando earthquake of February 9, 1971 is of interest.

The Building and Safety Department of the City of Los Angeles inspected 12,160 structures in the month following the earthquake using practially its entire force of 250 building inspectors. In Reference 5, this Department states as follows(SEAOC, 1971):

"The most important thing that the Department learned was that communications are the Department's most important asset in an emergency. The work accomplished was possible because of the telephone and the fact that, with the exception of the highly devastated area, the telephones continued to function. Had the earthquake been of a greater magnitude and the telephones unusable, the Department's efficiency would have been reduced to 10 percent. It is recommended that an emergency communication system, probably radio, be established for use by the Department. Improvement is needed to develop a procedure to make a quick location of the heavily damaged areas and an early evaluation of the overall damage. In this earthquake, it was 12-hours before the Department was fully aware of the magnitude of the problem it faced. The use of helicopters would certainly speed the Department's efforts in determining the extent of damage. There is a need to develop a better form for reporting statistical information on the first inspection. The Department received many requests from different agencies for statistical information which it was not prepared to provide, but which could have been provided had the inspection forms been properly formulated, this is a vital point. The disaster plan should provide emergency help. Fortunately, this Department was able to cope with this disaster because of its pre-disaster plan, and the four-day weekend following the earthquake, though at one point the Department was seriously considering a request to the Cities of San Diego and San Francisco for emergency help under the State's Mutual Aid program. Had the earthquake been of a greater magnitude and such outside help been needed, some plans for the use of inspectors who were not familiar with the local areas should have already been prepared and made ready for use. In a great disaster, the local building departments cannot expect local help because of the commitment of local structural engineers and architects to their local clients.

When one considers that the damage from this earthquake was moderate compared to what might clearly be expected in a major quake, it is obvious that planning for early inspections must include the organizing of governmental and private personnel on a state-wide basis. Such functions should be part of state Disaster Plans and should be well integrated with the plans of cities by the Federal Disaster Assistance Administration."

Even in California, where earthquake dangers are a daily concern, there has been no comprehensive state-wide approach to this problem. The lessons of the past have not yet been properly learned. Unless such preparation as has been discussed is implemented on a standby basis and updated, public safety will be compromised in the next major disaster.

Sec. 15.3 SELECTION OF INSPECTION PERSONNEL

The selection of inspection personnel involves a number of steps which are discussed in this Section.

15.3.1 QUALIFICATIONS

The following types of personnel will be needed for building inspection:

Professional Structural Engineers
Professional Mechanical Engineers
Professional Electrical Engineers
Architects
Construction Supervisors
Photographers
Laborers

15.3.2 RECRUITMENT SOURCES

The recruitment of inspection personnel may be handled through a number of organizations.

(A) PROFESSIONAL ENGINEERS. Professional engineers may be recruited through the auspices of the societies who represent such professions. Among these are the following:

American Society of Civil Engineers
National Society of Professional Engineers
American Consulting Engineers Council
Structural Engineers Associations (State Groups)
American Institute of Architects
American Society of Mechanical Engineers
American Society of Heating, Refrigeration, and Air Conditioning Engineers
American Institute of Electrical and Electronics Engineers
Society of American Military Engineers

Also, professional engineers may be recruited from corporations employing them in large numbers, such as oil companies, construction companies, and utility companies. Further sources for recruitment are governmental agencies--federal, state, and local.

(B) ARCHITECTS. Architects may be recruited from the American Institute of Architects and the Construction Specifications Institute.

(C) BUILDING INSPECTORS. Building inspectors and technicans may be recruited from associations of building inspectors and the Institute of Certified Engineering Technicians (sponsored by the National Society of Professional Engineers). The largest source of inspectors would be governmental agencies.

(D) CONSTRUCTION SUPERVISORS. Construction supervisors may be recruited from available personnel of the Associated General Contractors and various associations of builders and contractors organized within the various states and counties. It should be recognized, however, that contractors will undoubtedly be employed in the post-disaster period and may be unavailable.

(E) PHOTOGRAPHERS. Photographers could be organized on a local basis from professional associations.

(F) LABORERS. Laborers may be recruited from local union organizations.

(G) STUDENTS. Engineering students represent a relatively large, somewhat knowledgeable reservoir of active, motivated, informed but inexperienced assistants. Their possible role as helpers should not be minimized.

(H) AMATEUR RADIO OPERATORS. A listing of amateur radio operators and qualified citizens band operators should be prepared.

15.3.3 ENROLLMENT OF PERSONNEL

Enrollment would be best handled on a state level in order to facilitate the furnishing of personnel from areas outside that affected by the disaster, should such need arise. However, lists of personnel should be made up on a local area basis for the use of such areas to the extent that such personnel would be available at the time needed. A two-year enrollment period is recommended during which persons involved would agree to be on call for such service and to participate in training sessions and exercises for this purpose.

A sample recruitment form (Exhibit 15-2) developed and distributed to the membership of the Structural Engineers Association of Northern California resulted in 160 responses out of a membership of 773. The information on this form was designed to be transferred onto three computer punch cards for storage and subsequent printing and sorting. The data extracted and stored can be manipulated into alphabetical listings, geographical distribution (by zip code), listing as to age and capability, or any statistical parameter desired. A similar form could be developed on a state-wide basis for all types of personnel and could be distributed and collected through the sources mentioned in the foregoing subsections of this Chapter.

Enrollment could best be done under state auspices and could possibly invoke the state's broad powers of conscription under emergency conditions. Personnel would be enrolled as temporary state employees and be assigned to assist local government agencies who would be in charge of building inspection operations. Alternatively, personnel could be hired as consultants by either state or local jurisdictions, but this would be difficult to plan for in advance of need and would involve more difficulties in the area of liability of inspection personnel for their actions and recommendations during the emergency. In any event, the enrollment of inspection personnel must be accomplished by appropriate means to relieve the individual from such liability, since he will be called upon to render decisions which are necessarily made rapidly and without extensive investigation and analysis. Legislation is probably needed to accomplish this very-necessary protection.

The type of work envisioned by this Chapter does not lend itself to the nonpaid volunteer. Technical expertise of the highest caliber is called for and should be appropriately compensated. Accordingly, the rate or means of compensation should be agreed upon as part of the biannual enrollment procedure.

15.3.4 TRAINING

As was recommended for enrollment, training of personnel would also be best organized and administered by the state, in the interest of uniformity. Course material could be jointly prepared by technical societies and building inspection agencies as could the presentation of courses. Course content and frequency of sessions will depend upon the type of personnel concerned. Those charged with inspection of the large numbers of smaller structures need to be trained in the use of forms for operational procedures. One day annually would probably suffice for this type of training.

For professional engineers charged with investigating major structures, training courses would conceivably require four or five lectures sessions per year. All types of personnel would benefit from slide lectures or motion pictures of past disaster situations accompanied by lectures from knowledgeable people who could advise upon the best ways to accomplish inspections under the adverse conditions prevailing in such situations. Electrical and mechanical engineers will require special training.

The scope of training courses is outlined as follows:

(A) TRAINING COURSE FOR STRUCTURAL ENGINEERS. The training course for Structural engineers would include:

1. Mobilization Procedure
2. Equipment
3. Team Organization
4. Reporting Procedures--Use of Forms
5. On-site Evaluation of Structural Failures--Their significance with regard to continued occupancy.
6. Methods of On-site Determination of Structural Systems for Buildings without Benefit of Plans
7. Identification of Structural Failures
8. Hazard Identification--Adjacent structures, nonstructural materials and Assemblages
9. Temporary bracing methods
10. Estimating Methods for Determining Dollar Value of Damage

(B) TRAINING COURSE FOR BUILDING INSPECTORS AND ARCHITECTS. The training course for building inspectors and architects would include:

1. Mobilization Procedure
2. Equipment
3. Team Organization
4. Reporting Procedures--Use of Forms
5. Identification of Structural Failures
6. Instruction in the Lateral Load Resisting Systems Commonly Used for Dwelling Houses and Small Buildings
7. Hazard Identification

15.3.4(B) Cont.

8. Procedures for Evaluating Conditions of Utilities and Electrical Systems in Small Buildings

9. Estimating Methods for Determining Dollar Value of Damage.

(C) TRAINING COURSE FOR MECHANICAL AND ELECTRICAL ENGINEERS. The training course for mechanical and electrical engineers would include:

1. Mobilization Procedure

2. Equipment

3. Team Organization

4. Reporting Procedures--Use of Forms

5. Methods of On-site Determination of Mechanical and Electrical Systems for Buildings without Benefit of Plans

6. Shut-off Procedures

7. Identification of Hazards From Mechanical and Electrical Systems

8. Cost Estimating Methods

(D) TRAINING COURSE FOR OTHER PERSONNEL. The training course for other personnel would include:

1. Mobilization Procedure

2. Equipment

3. Team Organization

In addition to training courses, it is recommended that training exercises be conducted periodically--probably at least once during each enrollment period of two years. These should be conducted by the building inspection department in cooperation with the Disaster Relief Agency and other participating government agencies. These should be practical field exercises, to test all phases of mobilizing and carrying out the inspection procedure. This would best be done as part of a similated disaster in which all participating parties in disaster relief would respond. Such a field exercise should take place at least once toward the beginning of every two-year enrollment period and should be conducted by the State Disaster Relief Organization with the cooperation of all local agencies.

15.3.5 EQUIPMENT

Equipment lists for inspection personnel should be carefully planned and should be divided into personal and team equipment.

(A) PERSONAL EQUIPMENT. A list of personal equipment should be given to every participant, recognizing that access in or out of the area may be restricted for several days. Thus, each person should bring clothing and personal gear to sustain him for at least one week. It is assumed that the community and/or emergency relief organization would provide food and housing for these persons. The following list contains items which each person should bring to assist him in performing the work.

15.3.5(A) Cont.

1. Hard Hat
2. Knee Pads
3. Clipboard, Pencils, Erasers, etc.
4. Ten-foot Rule
5. Flashlight and Extra Batteries
6. Notebook
7. Battery-operated portable radio.

(B) ISSUE EQUIPMENT FOR EACH INSPECTION TEAM. Certain supplies can best be stored at the mobilization points for issue as needed, unless teams are organized in advance to bring the following necessary items:

1. Emergency Earthquake Damage Inspection Forms
2. Portable Communication Equipment plus Operating Instructions and Report-in Times Noted
3. Battery-operated Lantern and Extra Batteries
4. Eight-foot Folding Ladder
5. Plumb Bob and 200 Feet of Line
6. Carpenter's Level
7. Hand Level (Surveyor's)
8. Camera, Flash Bulbs and Film
9. Hand Tools--including claw hammer, hand saw, wire cutters, and wrecking bar
10. One-hundred Foot Tape
11. Street maps, with areas to be covered by each team and communication subcenters plainly marked.

(C) OPTIONAL EQUIPMENT. Certain optional equipment can be extremely helpful; such as:

1. Hand Calculators or Slide Rules
2. Tape Recorders, a Supply of Tapes, and Extra Batteries

(D) SUPPLIES. Inspectors should be supplied at the mobilization point with inspection forms and posting signs. Quantities of these must be made up in advance of need according to best estimates available. Transfer of such supplies between adjacent areas should be considered but not relied upon in total because of the possibility of restricted travel. If possible, mobilization points should be equipped with high capacity copying machines and ample supplies of copy materials.

15.3.5 Cont.

(E) RENEWAL OF STORED EQUIPMENT AND SUPPLIES. The need for periodic renewal of equipment and supplies stored at mobilization points must be anticipated, particularly for perishable materials such as batteries, film, and copy materials.

15.3.6 MOBILIZATION

Mobilization planning should include the items discussed in this Section.

(A) MOBILIZATION POINTS. Mobilization points should be established with the thought in mind that some may be rendered inoperable by the disaster, in which case mobilization in adjacent areas would be activated. Location of mobilization points is dependent on the physical makeup of the area involved and the organization of the disaster agencies involved. These points should be located where several forms of transportation are feasible (cars, trucks, water, air). They should also have radio communication capability for receiving and giving information as to number of personnel needed or available, where to transport, etc. A helicopter landing area near the mobilization point would be desirable in order to rapidly transport personnel to operating centers or other places of need.

A mobilization center outside the affected area, probably operated by the State Office of Emergency Services or its equivalent, should be established for the reporting of inspectors residing outside the affected area. For inspectors residing in the affected area mobilization points close at hand must be provided. These could be located adjacent to or in the Emergency Operating Center. In any event, transportation of mobilized inspectors must be arranged and ready. Preferably, in-area mobilization points would serve as the Emergency Operating Center for the building inspection process. If such a center also housed police, fire, and other vital municipal functions, communications and coordination of effort with these entities would be greatly enhanced. It should be noted that sturdiness of construction and capability to resist strong earthquake forces are very desirable attributes of the building which houses the Emergency Operating Center.

(B) LISTS OF MANPOWER. Lists of manpower for the building inspection function should be made on a state-wide as well as local basis in order to facilitate transfer of personnel to places of need. Such lists complete with addresses and telephone numbers should be maintained at local points of mobilization, and at municipal and county headquarters as well as at the State Department of Emergency Services headquarters. The lists should be updated not less frequently than every two years.

(C) NOTIFICATION. Inspection personnel should be instructed to respond automatically by reporting to their mobilization point without notification, if they have been unaffected by the earthquake and are located in or near the disaster area. Personnel from more remote areas should receive notification through their own building inspection department or jurisdiction. Ordinarily, telephone communication would be the best means for such notification. In case of widespread loss of telephone service, notification by various radio bands should be considered as alternatives.

(D) TRANSPORTATION. Personnel reporting from within the stricken area must use any means of transportation in operation, including personal vehicles if necessary. Personnel from remote areas would make use of their own cars and/or public transportation to bring them close to the area of need. Pre-arrangement for use of government vehicles, helicopters, and car pools should be made as part of the operation plan for this work.

(E) PASSES. A form of pass which will be familiar to state and local police should be issued in advance to all persons who have been accepted for this work. Extra passes in blank form should be available at points of mobilization for emergency use as required. Control of such issue should be maintained by the head of the Building Inspection Agency.

(F) INSPECTION TEAM ORGANIZATION. During the first few hours following an earthquake, the building department should by pre-arrangement inspect structures which are of vital importance to the health and safety of the community (fire and police facilities, hospitals, buildings for housing persons who are displaced from their homes, etc.). This function should be carried out by personnel of the building department with the assistance of private sector personnel who have agreed to be available for such emergency service. (As stated elsewhere in this Chapter, most local engineers in private practice could not be depended upon since they would be serving their own clients.) Pre-assignment of buildings to be initially inspected should be made in advance of need, considering proximity of buildings to the home or office of the persons to whom they are assigned, familiarity of the person with the building, etc. A system of reporting results of these initial inspections, assuming loss of normal means of communication and transportation, should be arranged in advance of need.

Organization of inspection teams will follow one or two days after the earthquake, and will vary with the type of area to be surveyed and types of structures which each team will be asked to inspect. Flexibility of assignments is necessary due to the uncertainty of the availability of specific persons at the time needed. To the extent that properly qualified persons do report for service, they could be pre-assigned to inspect certain large, or critical, structures with which they are familiar.

Those assigned to the inspection of dwelling houses and smaller structures can operate as one- or two-man teams. Ideally, larger structures would require a team consisting of one or more structural engineers, a mechanical engineer, an electrical engineer, and supporting staff such as construction laborers, technicians, photographers, etc. However, it should be recognized that, under practical emergency conditions, it may not be possible to attain such a balanced team.

In addition to inspection teams, the Operating Center should be manned to administer programs, obtain and issue supplies, process data as it is collected, and give technical advice to the inspection personnel.

Sec. 15.4 PROCEDURES FOR INSPECTION

Several basic procedures necessary to set the early inspection operation in motion are described in this Section.

15.4.1 ESTABLISH AREAS OF DAMAGE

It is difficult to quickly determine the areas of severe damage. Twelve hours passed before the full extent of the damaged area was known following the initial shock of the San Fernando earthquake of February 9, 1971. The use of helicopter overflights is acknowledged to be the best means of obtaining a quick determination of area of damage. (Los Angeles County Earthquake Commission, 1971) Also, photo reconnaissance flights should be initiated as soon as possible to aid administrators in determining the degree of damage and potential hazards (Federal Disaster Assistance Handbook for Government Officials, undated). Planning for overflights should be a function of the State Office of Emergency Services in cooperation with local agencies. A second means of evaluation is so called "windshield surveys" which are done by driving through the areas on a block-by-block basis. Potential problems of access due to heavy damage, road blocks, etc., as well as the obviously less-rapid accumulation of data make this procedure much less desirable than overflights.

15.4.2 CENTRAL CONTROL GROUPS

The central control group would be the building inspection agency having jurisdiction over the damaged area. Possibly the area will encompass more than one jurisdiction, in which case more than one central control group would be estalished. It would be the task of the State Office of Emergency Services to coordinate efforts of each and provide for mutual aid between the areas. The central control group would perform the following functions:

1. Establish communication with central disaster relief organizations.
2. Mobilize office force and inspection teams.
3. Retrieve equipment and supplies from storage and issue to inspection teams.
4. Set up office.
5. Arrange transportation.
6. Arrange communications.
7. Arrange feed and housing of personnel.
8. Process inspection reports.
9. Respond to citizens requests for inspections.
10. Issue-statistical data to agencies and news media having need for such
11. Conduct inspection parties through damaged areas.
12. Other.

15.4.3 CLOSING OFF OF DAMAGED AREAS

This function would be the responsibility of the police. The building inspection department would need to be informed of the location of such boundaries and would need to make arrangement for passage of its personnel and their vehicles through the barricades.

15.4.4 SELECTION OF TEAMS

Inspection teams would be formed from the inspection personnel as they report for duty, making the best use of persons available. Engineers who are familiar with specific structures or types of structures should be assigned to teams investigating such buildings.

A priority of inspection should be established as part of the Disaster Plan for the community, placing top priority on those structures which are most urgently needed for use by the community.

15.4.5 TRANSPORTATION OF CREWS

Planning for transportation should include arrangements for passage of inspectors through barricaded boundaries of the stricken area. A form of pass familiar to the police and military forces should be issued. This could best be handled by the state with provision for local issuance on a limited basis after a disaster has struck.

Transportation of inspection teams to operating points will depend upon the conditions of and availability of public transportation and normally used private and public vehicles. Plans should include pre-arrangements with cab and auto rental companies to use their vehicles on an emergency basis.

15.4.6 COMMUNICATIONS WITH INSPECTION TEAMS

This is a vital need which should receive high priority in advance planning assuming that the telephone system is inoperable. An emergency radio system for use by the building inspection department with mobile two-way units for each team, would be the best means of communication. If such is not available, the following means of communication might be considered:

1. Taxicab two-way system
2. Citizens band radio
3. Walkie-talkie
4. Amateur short-wave radio stations
5. Private cars
6. Military jeeps, trucks, etc.
7. Messengers on foot

Sec. 15.5 EVALUATION OF STRUCTURAL DAMAGE

In the event of a major earthquake, many buildings will be damaged to varying degrees and some will collapse. The overriding consideration for continued occupancy of the buildings is the integrity of the structural frame after the earthquake. Earthquake damage to the frame is dependent upon the type of lateral force resisting system, age and construction type of the building, duration and severity of ground shaking, and associated ground movements, such as sliding, liquefaction, and ground consolidation.

Since severe aftershocks may occur after an earthquake and further weaken a damaged structural frame, it is of paramount importance to make an immediate inspection of the building frame and assess the degree of damage and the potential of the frame to resist further shaking. Other components of the building such as nonstructural walls, ceilings, lights, and exterior ornamentation may be damaged, but the primary concern to the safety of the occupants of the building is the integrity of the structural frame.

Once inspection teams have arrived on the building site, the type of lateral force resisting system must be identified and the building examined to determine if the structural frame has been damaged sufficiently to warrant posting the building unsafe for occupancy. This process will entail the removal of finishes in some case to view and evaluate the structural frame.

15.5.1 EMERGENCY EARTHQUAKE DAMAGE INSPECTION FORM

This form (see Exhibit 15-4) is designed to be completed quickly with a minimum of writing. It is designed to gather data on structural and nonstructural damage as well as damage to mechanical, electrical, and plumbing sustems, and soil and site conditions for the average building. The "Emergency Earthquake Damage Inspection Form" is designed so the data shown on the form can be transferred onto two computer punch cards if the governmental body chooses to use electronic data processing techniques. The form is equally useful if the governmental body chooses not to use electronic data processing.

Following a major earthquake, many governmental bodies are involved, each desiring certain kinds of information. The advantage of transferring the data obtained from the Earthquake Damage Inspection Form onto computer cards is that the mass of data can be

15.5.1 Cont.

stored, retrieved, and sorted in any manner desired by the many jurisdictions involved. For example, total dollar damage or a listing of all damaged buildings within a given zip code area can be obtained. Additionally, data processing could be done outside the damage area to avoid confusion or if electricity or data processing equipment are lacking. The Earthquake Damage Inspection Form is equally applicable to large and small buildings. The investigator fills in the blank boxes with the appropriate words and numbers which can then be turned over directly to a keypunch operator.

It is suggested that the Emergency Earthquake Damage Inspection Forms be made up in triplicate with carbon backing, and that the investigator turn in the first and second copy at the end of each day to the building department, retaining the third copy for this own records. Forms should be transferred to a central operations center for tabulation and safe storage on a daily basis.

15.5.2 PHOTOGRAPHS

It is recommended that instant-process photographs be taken of major and severely damaged buildings and their components. These photographs should be identified on the back with the building serial number and address. These photographs will assist the supervisors in the emergency operations center to properly understand and assess the degree of damage to a given building.

15.5.3 POSSIBLITY OF ADJACENT BUILDINGS COLLAPSING ON BUILDING BEING INSPECTED

The investigator must constantly be aware that the building he is investigating may be adjacent to a building either ready to collapse or with serious potential hazards, such as parapets, ready to fall on the adjacent building during an aftershock.

15.5.4 RATING OF BUILDINGS FOR HAZARD AND CONTINUED OCCUPANCY

In assessing the degree of building damage, investigators will have to exercise their best judgement as to the severity of the damage to the building and its corresponding risk to the occupants. This assessment will require that the investigators carefully define the lateral force resisting system and decide after careful inspection of the system whether the building is safe to occupy. Such examination may require that portions of wall finishes and ceilings be removed to examine the building frame and its connections. The final rating of the building will depend largely on the expertise of the investigator. Thus, careful consideration should be given by the emergency supervisor as to the qualifications of an investigator sent to a given building.

15.5.5 POSTING OF BUILDINGS

After the inspection team has investigated a building, the building should be posted with one of the following distinctive cards:

Color Posted	Hazard Category
1. Green	No significant damage - unlimited entry
2. Yellow	Minor significant damage - limited entry
3. Red	Unsafe; major damage - entry prohibited

15.5.6 ORDINANCE

The local government body should enact an enabling ordinance related to emergency inspection of buildings. (See Exhibit 15-3.) This ordinance would prohibit entry into a building so posted, and would be backed by the law enforcement agency. This ordinance would also make it illegal to remove such postings and establish appropriate legal penalites for so doing. Such legislation has either been proposed or enacted by some local municipal governing bodies in California, but for uniformity one state-wide law would be better.

Sec. 15.6 EVALUATION OF NONSTRUCTURAL DAMAGE

Buildings can sustain severe nonstructural damage with or without damage to the building frame. Examples of nonstructural damage are broken glass, shattered partitions, fallen or damaged ceilings, fallen light fixtures, dislodged mechanical and electrical equipment, broken plumbing lines and water heaters, broken gas and water lines, and elevators and counterweights coming out of their guiderails. This nonstructural damage is often associated with excessive movement or failure of the building frame, but in some cases may result from shaking without failure of the building frame.

The investigator will have to exercise his judgement as to whether the nonstructural damage represents a hazard to occupants. Obviously, when nonstructural damage results from damage or failure of the building frame, it represents a hazard to be considered with other sources. However, if the nonstructural damage does not result from damage to a building frame, then it may or may not represent a severe hazard to occupants. Such hazard, or lack of hazard, must be evaluated by the investigator.

15.6.1 EMERGENCY EARTHQUAKE DAMAGE INSPECTION FORM

The form in Exhibit 15-4 may be used for nonstructural damage with the appropriate boxes filled in by the investigator.

15.6.2 PHOTOGRAPHS

Photographs of nonstructural damage should be taken where such damage represents a hazard to building occupants. Examples of this type of hazard are ceilings ready to collapse or large panes of broken glass which could be blown in by wind or loosened by aftershocks.

Sec. 15.7 EVALUATION OF AUXILIARY SYSTEMS

Damaged or inoperative service systems supplying water, gas, electricity, sanitary sewers, etc., to the building may render a structure unusable or dangerous as a more dramatic and evident collapse of a structural or architectural finish system. The complexity and number of service systems in any one structure may vary widely. In addition, the critical need for certain classes of structures to be returned to operation as soon as possible makes early evaluation of the auxiliary systems almost mandatory. The training program should explain and describe the various common service systems and the location of various control points, so that potentially hazardous conditions may be recognized and corrective measures taken promptly by the first investigator.

Buildings with various classes of auxiliary systems and their order of importance after an earthquake may range as follows.

15.7.1 HOSPITALS

The hospital complex presents the greatest difficulty to the investigator due to the number, complexity, and interdependence of the various service systems installed in the building. In addition, potentially hazardous substances and gases must be detected and neutralized. Because of these extensive and varied requirements, the inspection team dispatched to examine such facilities after an earthquake should be a special, "blue ribbon" group composed of a highly qualified structural engineer, a mechanical engineer, and an electrical engineer. In the records at the emergency operating center, the names, current addresses and telephone numbers of the permanent operating personnel of the various hospitals in the area of jurisdication should be permanently on file. When a special team is dispatched to a specific hospital, arrangements should be made so that one or more of the most qualified hospital operating personnel accompanies the team. During the inspection, consideration should be given not only to the physical state of the facility and the practicability of its continuing operation, but also a very careful investigation should be made of the possible escape of dangerous gases and of liquids or solids which may combine to form dangerous gases if thrown from the shelves during an earthquake.

15.7.2 POWER STATIONS, TRANSFORMER STATIONS, PUMPING STATIONS FOR WATER AND SEWAGE, COMMUNICATION FACILITIES, ETC.

The quick restoration to service of these facilities is probably equal in importance to the restoration of hospitals. Most of these facilities are owned and operated by municipalities or public utilities with competent staffs. It is assumed that most of these agencies have developed emergency plans which are presumably operative, and therefore it is also assumed that immediately after the first earthquake shock the necessary investigative and repair work for such facilities would be carried out by the operating agency directly concerned.

15.7.3 STANDBY FACILITIES, SUCH AS SCHOOLS WITH AUDITORIUMS, GYMNASIUMS, OR CAFETERIAS.

In the State of California, the design and construction of school buildings has been carefully regulated since the 1930s by the Office of the State Architect, so that the number of school structures which could reasonably be expected to suffer severe damage is relatively small. In other parts of the United States, where the seismic hazard is less, this statement is not necessarily true, and it is probable that the service systems of these buildings could be expected to undergo relatively heavy damage. The service systems in this type of structure are usually simple and few in number, and their potential for creating hazardous conditions is therefore comparatively small. In the case of school buildings, the school system in most political subdivisions is well organized, and the names, addresses, and telephone numbers of the chief custodian of each individual school could be kept currently on file in the Emergency Operating Centers. It is believed that the inspection team sent to this class of structure need not be as broadly based. It is believed that if the team has a man qualified in the appraisal of structures he, working with the custodian, would be capable of assessing the damage and hazards in the service systems. In the State of California, the Office of the State Architect will undoubtedly be conducting concurrent investigations of various school buildings and consideration should be given to working out arrangements whereby this particular class of investigative work is not duplicated.

15.7.4 FOOD WAREHOUSES

There is a need for early investigation of food warehousing facilities to ascertain their capability for providing safe storage or whether vital food stocks should be immediately removed.

15.7.5 OFFICE BUILDINGS

The service systems of modern office buildings present problems of varying degrees. In general, the services provided are relatively simple and hazardous gases and substances are usually not present.

The physical complexity of the building may present major difficulties for inspection. Mechanical equipment in low-rise office buildings is usually concentrated either in the basement or on the roof, and the power-supply, ducting and piping systems going to and from these areas are relatively simple and easy to investigate. The stability of the equipment after the earthquake is of major concern; i.e., if the equipment has broken loose from its supports, it may overturn and/or go through the roof, and/or start fires.

Very high modern office buildings afford a more complex problem. They very often have separate mechanical equipment floors located at points such as midheight or third point heights in the building. The power supply, ducting, and piping systems may be quite large and complex, and difficult to inspect. Also, the mechanical equipment itself is apt to be much larger and heavier than in a low-rise building and thus could present a much greater hazard if it should become dislodged from its base or overturned.

The operating personnel for a major office building are quite highly skilled and well organized. It is recommended that a current file be kept of these people, and that they be made a part of the inspection team dispatched to the specific building. In addition, if the structure to be examined is a major one, it is also recommended that the investigating team be of the same makeup and caliber as the "blue ribbon" team mentioned above in the case of hospitals.

15.7.6 MANUFACTURING PLANTS.

The size and complexity of service systems provided for manufacturing plants vary widely. It is believed that the standard investigation team should be capable of inspecting and judging hazards connected with what might be called an "average" plant. It is recommended that in those cases where dangerous or toxic materials are present or potential hazards of fire or explosion are great, current information as to the type of risk should be on file in the Emergency Operating Centers, and the investigation team should be provided with a list of critical items to check at the time of the inspection. In preplanning for the investigative work, it is recommended that the agency work with the fire department of the political subdivision in which the plant is located because it is probable that the fire department maintains up-to-date files on potential hazards of this nature.

15.7.7 APARTMENT BUILDINGS, INDIVIDUAL RESIDENCES

The services provided in residential buildings are relatively simple and the major components of the systems are grouped in one area, usually the basement. This generally affords easy access providing the building has not collapsed. Because of the large number of these types of structures throughout the city or other municipality, it would be impractical to assign more than one qualified person to examine an individual building. The training program should explain and describe the various common service systems and the location of various control points, so that potentially hazardous conditions may be recognized and counteractive measures taken promptly by the first investigator.

15.7.8 ELEVATORS

In past earthquakes, the performance of elevators in general has been rather poor. In many instances, counterweights have jumped from the guiderails, guiderails for the cab have buckled, the hoisting machinery has moved on supports, and elevator shafts have gone out of plumb as the result of permanent building drift. Because the elevator controls are complex and the potential hazards associated with their operation are great, it is recommended that the preliminary inspection by the first investigative team be confined to a relatively cursory inspection of easily observable items, and that the elevators be posted as nonoperable until qualified personnel from the elevator company have checked them. Again, the order of importance would place the elevators in hospitals at the top of the inspection list and, from a practical use standpoint, those in high-rise office buildings next.

15.7.8 Cont.

Exhibit 15-4, Page 4 of 4, Supplemental Form, presents a check list of items to be investigated at the time of the first inspection. While no list can be all-inclusive, it is intended that this list will act as a basic directive for the team's work.

Sec. 15.8 ON-SITE SOIL AND FOUNDATION CONDITIONS

Changes in the condition of on-site foundation materials may seriously affect the safety and stability of a structure being inspected. In addition, the type, arrangement, etc., of materials underlying the building greatly influence the input of earthquake motion into the building. Before investigating the various buildings and structures assigned to them, inspection teams should obtain some knowledge of the general characteristics of the soil in the area in which they will be working. Such studies should be part of the training course and will prove invaluable as a guide during inspections. Upon entering the area, and especially upon first approaching the specific building or structure to be investigated, the inspection team should endeavor to examine the immediate area for the more apparent surficial evidence of movement of underlying soils. During the second inspection, more time and care can be devoted to this part of the work.

The types of soil movement which may manifest themselves are discussed in a grossly simplified manner in the following sections.

15.8.1 SLIDING

Sliding may reasonably be expected to occur in an area where the natural grade is relatively steep and the top soil layers are underlain by differing materials or lubricating layers, such as saturated clay. Such movement may manifest itself in surface breaks in the street or sidewalk. These breaks may exhibit vertical, horizontal, or lateral movement. Similar breaks or large-scale cracks in basement walls or floor slabs on grade may also provide evidence of sliding. Slides may also occur in relatively flat terrain and may be very large; a classic example of this is the slide that occurred in the area under and around the Juvenile Hall during the San Fernando Earthquake.

15.8.2 FAULTING

Manifestations of surface breaks due to faulting in general are similar to those discussed for sliding.

15.8.3 SOIL LIQUEFACTION

Soil liquefaction may occur if certain soil types are present and the water table is close to the surface. The effects of liquefaction may be quite dramatic and drastic and may result in large-scale settlement of a building, or large-scale tilting and inclination of a building as a whole. Classic examples of this type of damge occurred at various buildings in Niigata, Japan. Liquefaction may also evidence itself at the ground surface by the formation of numerous small "sandy boils" or vents from which water may have flowed, or may still be escaping.

15.8.4 COMPACTION OR CONSOLIDATION DUE TO SHAKING EFFECTS

Compaction or consolidation may occur under footings of braced frames or of shear walls because of the overturning forces generated. This may result in "rocking" of the building, with residual out-of-plumbness of some parts.

15.8.5 LATERAL COMPRESSION OR SLIDING

Lateral compression or sliding of the materials surrounding or underlying bracing frames or shear wall footings may result from translational movement of the building footings. Such lateral displacement may cause recognizable ruptures in the walls or floors, or in rupture of foundation ties.

15.8 Cont.

15.8.6 LATERAL AND FLEXURAL FAILURES OF PILING

Failure of the foundation piling due to bending or shear forces caused by movement of the soils surrounding the piling may occur if there are soil conditions conducive to such deformation. Because the piling is beneath the building, it may be impractical to determine whether this type of damage has occurred especially where the building covers the entire site. In the case of a more open type structure, such as a bridge foundation, piled retaining wall, etc., such effects may be evident at the surface. The investigator should evaluate the surface evidence at the site and if unable to come to a conclusion should classify the building or structure for future investigation by a soils engineer.

Sec. 15.9 TSUNAMI AND SEICHE EFFECTS

Areas subject to tsunami and seiche disturbances are located close to large bodies of water. Prior occurrences of such phenomena will undoubtedly be a matter of record when the investigating team enters the area. Structural effects due to tsunami will usually be evident, the structure having been subjected to lateral forces due to water movement, and perhaps also due to the lodging of debris against the structure. The structure may exhibit signs of incipient collapse due to the very large forces imposed or, in the case of frame buildings, may have been lifted off the sill anchor bolts or the floors may have been lifted off supporting bearing walls.

In wood frame buildings the presence of silt-laden water may result in the deposition of mud in wall stud spaces. While this situation will not lead to immediate collapse, it could have a deleterious effect on the useful life of the building due to failure of the studs and sill plates.

In the case of seiche-like disturbances the rise and fall of the water level may not be accompanied by a noticeable and dramatic lateral movement of the water, but damage to the structure may be similar to that from a tsunami.

Undermining and scouring of spread footings may occur due to either type of disturbance. Damage to building services may be quite severe and damage to outside sewer and power connections may be very widespread so as to render facilities inoperable even if the structure examined is relatively undamaged. The investigative team should endeavor to note such damage so that its location may be immediately transmitted to repair crews of the various public utilities involved.

Sec. 15.10 CONTROLS AS A RESULT OF INSPECTION

The procedures outlined in the foregoing sections have been designed to fill the specific need for a quick, preliminary reconnaisance-type survey; the main purpose being to determine quickly what structures can be safely reoccupied and to make cost estimates of damage. Of necessity, the inspection results and the conclusions therefrom will be somewhat cursory. Buildings determined to be unsafe at this time would be immediately posted to establish that they are not to be re-entered.

15.10.1 REINSPECTION

Because of the drastic consequences to the owner and tenants from the assessment that a building must not be occupied, it is necessary that a routine reinspection of all buildings first classified as unsafe for occupation be carried out after a relatively short time. In addition, relatively large aftershocks are almost certain to occur in the period immediately following a major earthquake shock and further damage may occur in already-weakened buildings.

15.10.1 Cont.

It is therefore recommended that in the routine procedures for investigation, a follow-up be required wherein inspection teams are again sent out after a period of one week to reinspect all structures previously classified as marginal in the first inspection and all those previously classified as unsafe to occupy. These second inspections should be performed more meticulously and completely and in a more deliberate manner because of the lesser urgency. The teams performing these inspections should be more highly qualified than it would be practical to attain in all cases in the initial inspections. Preferably such teams should be comprised of licensed engineers with special experience and qualifications in the design of buildings. During the second inspection more information will be available from the first inspection report about the nature and condition of the building or structures to be investigated. Therefore, the type of equipment furnished to the investigators can be more complete, although it is believed that in general the equipment set up for the first inspection will suffice.

Because of the better knowledge of the task to be performed and because of the increased type and scope of inspections to be performed in the second inspection, licensed mechanical and electrical engineers should be recruited and on call to render judgements and make recommendations on the condition of the various types of damaged mechanical and electrical systems.

Because of the more intensive nature of the inspection to be carried out, a laborer equipped to remove finishes, etc., should accompany the inspecting party.

15.10.2 RECORDS

In order to justify the continued application of the original hazard rating or a change of rating resulting from the second investigation, a Record of Reinspection should be filed with the agency in charge. Suggested forms for this Record are shown in Exhibits 15-5 and 15-6 through 15-9. During the reinspection, a more exact estimate of repair should be made by the investigator(s). It is suggested that the forms and repair estimates be made out in triplicate, two copies being turned in to the supervising agency at the end of each day, with the individual inspector in charge of the investigating team keeping one for the teams' records. Copies or transcripts of these forms and reports should be sent each day to the police and fire departments, and to the participating state and federal disaster agencies for their use.

The forms in Exhibits 15-6 through 15-9 have been designed to act as reminders of the various critical items to be inspected in the structural, mechanical and electrical systems, and architectural finish parts of a building so as to provide a basis for arriving at a final opinion as to the advisability of reoccupying the building. Forms for structural items have been subdivided into types of building materials and the main structural elements offering lateral force resistance. The critical components of these systems and materials are then listed to be inspected.

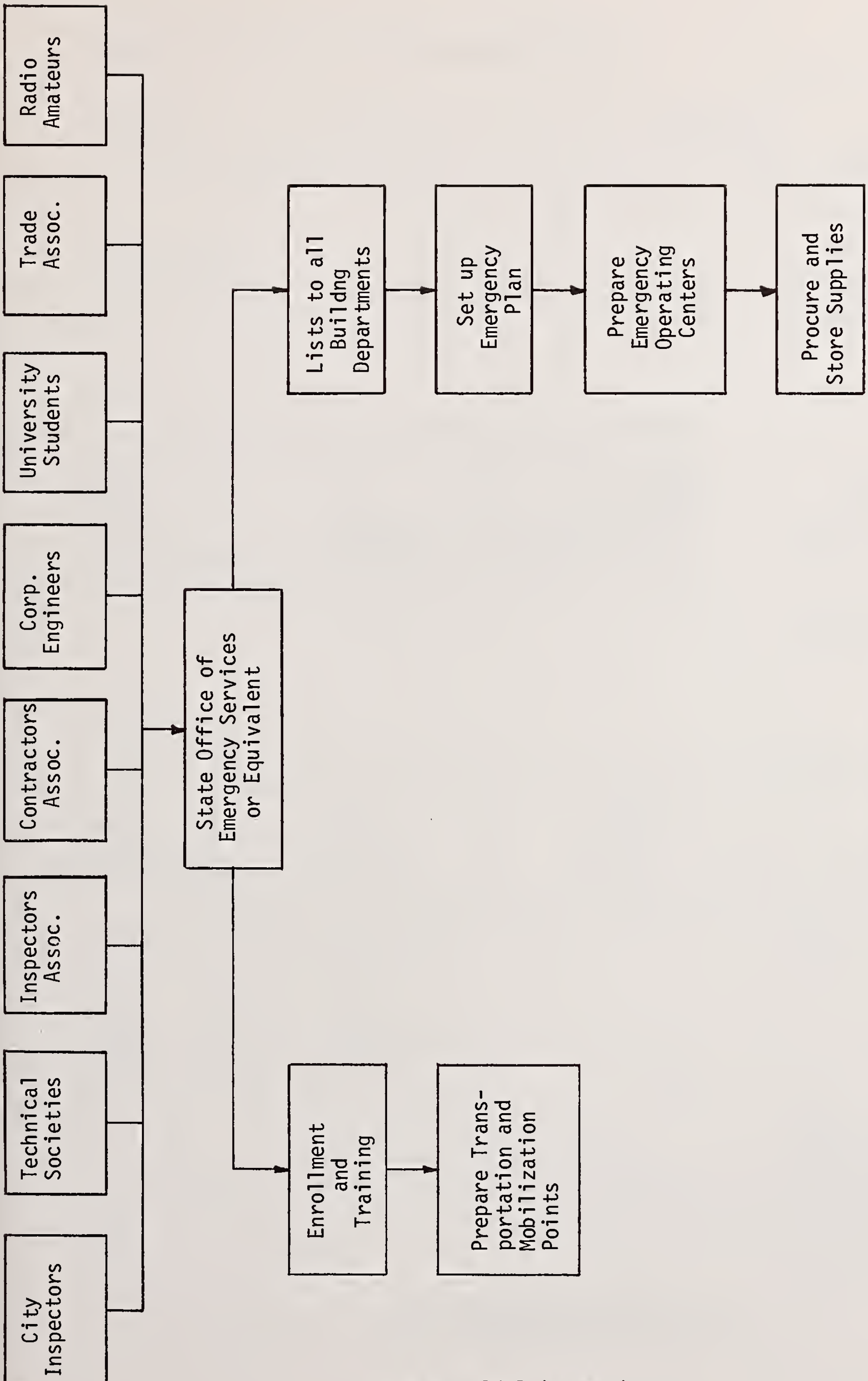
The forms may provide a legal basis for defense should action for damages be undertaken by a disgruntled owner of a condemned building.

For nonstructural and building service systems, the components are so subject to variation and intermingling that only the more fundamental and potentially hazardous items are listed.

15.10.3 REPAIRS

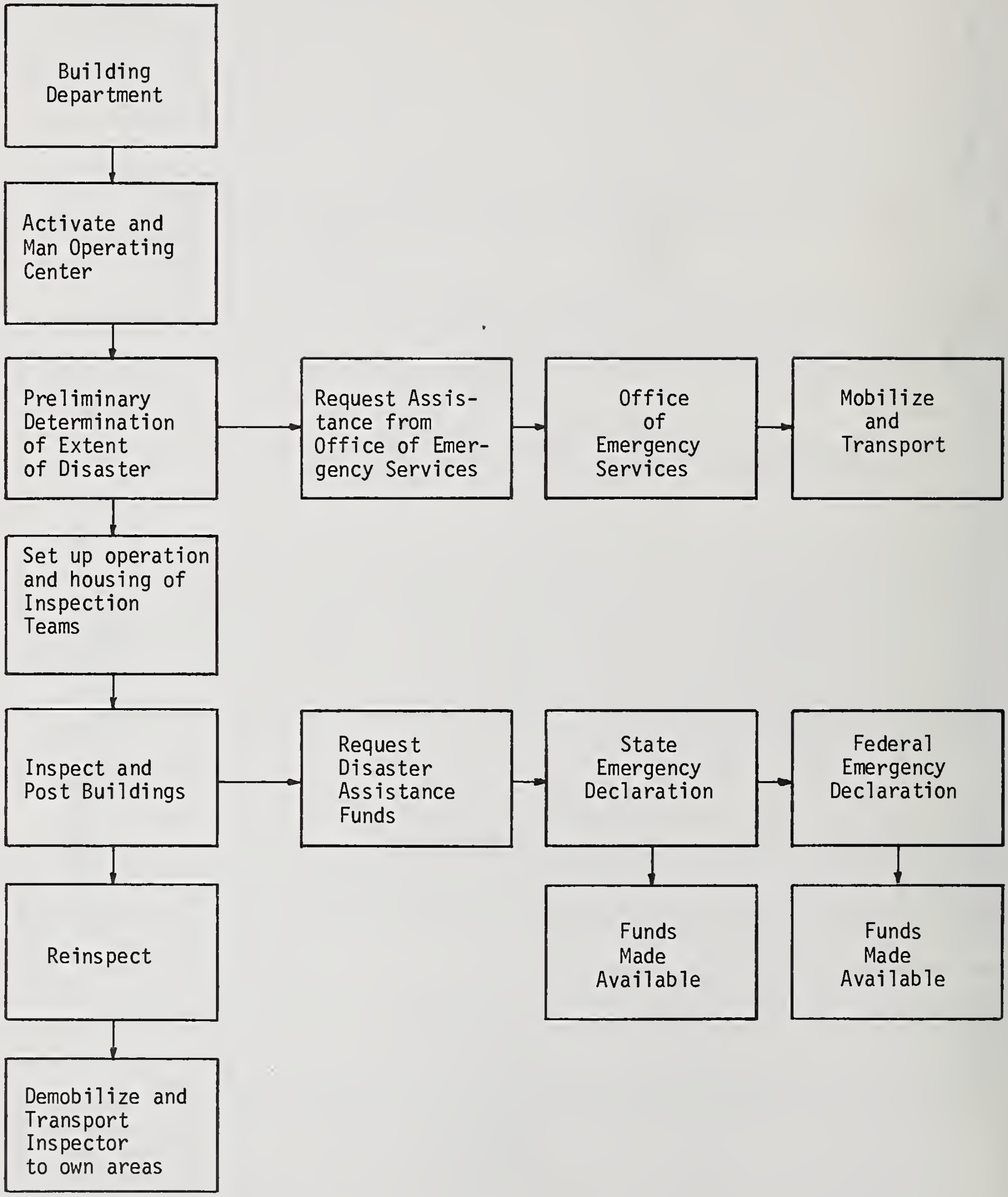
The requirements for adequate strengthening of damaged buildings and the process whereby these repairs are made and approved by the building department are are treated in Chapters 13 and 14.

15 Cont.



ORGANIZATION CHART
PRE-EARTHQUAKE - PREPARATION PERIOD

EXHIBIT 15-1 (1 of 2)



ORGANIZATION CHART
EMERGENCY POST-EARTHQUAKE PERIOD

EXHIBIT 15-1 (2 of 2)

EARTHQUAKE DISASTER PREPAREDNESS QUESTIONNAIRE

LAST NAME FIRST NAME

1 22

HOME ADDRESS

23 44

CITY

45 59

ZIP CODE

60 64

TELEPHONE (INCL. AREA CODE)

66 75

IDENTIFICATION (DO NOT FILL IN)

77 80

OFFICE NAME

1 22

OFFICE ADDRESS

23 44

CITY

45 59

ZIP CODE

66 64

TELEPHONE (INCL. AREA CODE)

66 75

IDENTIFICATION (DO NOT FILL IN)

77 80

EXHIBIT 15-2 (1 of 3)

NAME OF PERSON WHO CAN CONTACT YOU

AT ANY TIME

LAST NAME FIRST NAME

1 16

ADDRESS

17 33

CITY

34 48

TELEPHONE (INCL. AREA CODE)

49 58

EXPERIENCE: IF YOU HAVE HAD EXPERIENCE IN ANY OF THE FOLLOWING TYPES OF BUILDINGS, ENTER THE NUMBER 1 IN THE APPROPRIATE BOX:

HOSPITALS		59
OFFICE		
EDUCATIONAL		
INDUSTRIAL		
COMMERCIAL		
HIGH RISE		64

EXHIBIT 15-2 (2 of 3)

LICENSES: ENTER THE NUMBER 1 IN THE BOX OPPOSITE THE LICENSES YOU HOLD.

CIVIL	☐	65
STRUCTURAL	☐	
OTHER	☐	67

AGE GROUP: ENTER THE NUMBER 1 IN THE BOX OPPOSITE YOUR AGE GROUP.

20-30	☐	58
31-50	☐	
50 +	☐	70

TRANSPORTATION: ENTER THE NUMBER 1 IN THE BOX IF YOU CAN PROVIDE YOUR OWN TRANSPORTATION, LEAVE BLANK IF NO. ☐ 71

TRAVEL: ENTER THE NUMBER 1 IN THE BOX IF YOU ARE WILLING TO TRAVEL TO OTHER CITIES, LEAVE BLANK IF NO. ☐ 72

PHYSICAL CAPABILITY: RATE YOUR CAPABILITY TO DO PHYSICAL WORK BY ENTERING THE NUMBER 1 IN THE APPROPRIATE BOX.

STRENUOUS	☐	73
MODERATE	☐	
OFFICE	☐	75

IDENTIFICATION (DO NOT FILL IN)

☐☐☐☐
77 80

UPON COMPLETION OF THIS FORM, PLEASE RETURN TO
STRUCTURAL ENGINEERS ASSOCIATION OF NORTHERN CALIFORNIA
171 SECOND ST., SAN FRANCISCO, CAL. 94105

EXHIBIT 15-2 (3 of 3)

SUGGESTED ORDINANCE

RELATING TO EMERGENCY INSPECTION OF BUILDINGS

SECTION 1. DECLARATION OF FINDINGS. The City Council of the city of__ ______ ____________________ does hereby find and declare:

(a) That in the event of an enemy attack, explosion, fire, riot, windstorm, flood, tidal wave, earthquake, landslide or other disaster it is possible that a large number of buildings within the city be damaged.

(b) That it is in the best interest of the public that all buildings within the areas where damage has occurred be inspected to determine whether such buildings are safe for occupancy.

(c) That it is in the best interest of the public to make such inspections as soon as possible, and in order to accomplish such inspection, it is necessary to use all available resources of manpower and equipment.

SECTION 2. The provisions of this ordinance shall become effective immediately whenever a local peril, local emergency, local disaster or state of extreme emergency or state of disaster have been declared by proper authority and shall continue in effect until terminated by a resolution of the City Council.

SECTION 3. APPLICATION. All buildings within the entire city or within any area designated by the Building Official or other proper authority shall be vacated immediately and shall not be re-occupied until such buildings have been inspected and approved for occupancy. The Building Official may, because of the nature of the construction and occupancy of certain types of buildings, permit occupancy without inspection.

EXHIBIT 15-3 (1 of 4)

SECTION 4. RIGHT OF ENTRY. The Building Official and his authorized representatives may upon showing proper credentials inspect any building within the city at any time. No search warrant shall be required to make such inspection. No person shall prohibit, prevent or obstruct the Building Official in inspecting any building. The Building Official or his authorized representative may remove portions of wall or ceiling finish or other material in order to inspect concealed portions of the structure.

SECTION 5. DEPUTIES. The Building Official may enlist volunteers or require persons with specialized training and experience in the field of building design or construction to assist him in inspecting buildings. Such persons who have been required to perform inspections shall be compensated as specified by the City Council.

SECTION 6. LIABILITY. The Building Official and his authorized representatives including all volunteers or persons impressed into service shall not render himself personally liable for any damage that may accrue to persons or property as a result of any act required or by reason of any omission in the discharge of his duties. Any suit brought against the Building Official or his authorized representative because of an act or omission performed by him in the enforcement of this ordinance shall be defended by the city.

SECTION 7. UNSAFE BUILDING. Any building or portion thereof which in the judgment of the Building Official or his authorized representative after inspection is not safe for occupancy shall be vacated forthwith and shall not be re-occupied until the building or portion thereof is made safe and re-occupancy is approved by the Building Official.

EXHIBIT 15-3 (2 of 4)

SECTION 7. (Continued)

Any building or portion thereof which in the judgment of the Building Official or his authorized representative after inspection constitutes an immediate hazard to the health, safety or general welfare of the public, or endangers private or public property may be abated by immediate demolition and removal, and the Building Official is hereby authorized to take whatever steps as may be necessary to accomplish summary abatement.

The Building Official or his authorized representative may permit limited entry into unsafe buildings for the purpose of rescue, salvage or temporary repairs. Such requirements shall be enforced by the Police or other authorized security groups.

SECTION 8. POSTING. The Building Official or his authorized agent, after inspection, may post notices in a conspicuous place at the entrance to each building. The notices may prohibit entry, permit limited entry into unsafe buildings, or indicate that the building is safe for occupancy. No person shall enter any buildings into which entry has been prohibited. Only persons authorized by the Building Official or other proper authority shall enter any building which has been permitted limited entry. No person shall remove or deface any signs posted by the Building Official or his authorized representative.

COLOR POSTED	HAZARD CATEGORY
1. Posted Green	No significant damage - unlimited entry.
2. Posted Yellow	Minor significant damage - limited entry.
3. Posted Red	Unsafe - major damage - entry prohibited.

SECTION 9. REPAIRS. The provisions of the Building Code, Plumbing Code, Electrical Code and Heating and Ventilating Code relating to the issuance of permits and

EXHIBIT 15-3 (3 of 4)

SECTION 9. (Continued)

collection of permit fees are hereby modified as follows. The Building Official or his authorized representative may issue permits without the submission of plans, specifications or permit fees for the following:

(a) For repair to wood-frame structures where the damage is not serious and where repairs can be made with the same methods of construction and material used in the original building.

(b) For repair of interior non-structural members not required to be fire-resistive.

(c) For repair of electrical, mechanical and plumbing systems where the repair work will be done substantially in the same manner as the original construction.

(d) For the installation of temporary bracing or shoring.

EXHIBIT 15-3 (4 of 4)

CODERS KEY AND INSTRUCTIONS FOR CODING "EMERGENCY EARTHQUAKE DAMAGE INSPECTION FORM"

I. GENERAL

1. The form is designed so that the data can be punched up on two standard IBM punch cards.
2. All boxes are to be filled with the proper number, not a check mark. Insert a zero (o) if "Not Applicable".
3. The numbers under the boxes are columns on the IBM standard punch card (80, per card), and the building serial number must be filled in two places, at the top and at item E. This must be done as this is the only link between the two IBM cards.

EXHIBIT 15-4 (1 of 4)

EMERGENCY EARTHQUAKE DAMAGE INSPECTION FORM

BUILDING SERIAL NO. [ ][ ][ ][ ] (1–4) INVESTIGATOR CODE NO. [ ][ ][ ] (5–7)

DATE [ ][ ][ ][ ] (8–11) TIME [ ][ ] (12 13) AM (1) PM (2) [ ] (14)

A. BUILDING LOCATION: ADDRESS [ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ] (15–29)

ZIP CODE (LAST 3 NUMBERS) [ ][ ][ ] (30–32) SIDE OF STREET: N(1), S(2), E(3), W(4) [ ] (33)

STREET INTERSECTION [ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ][ ] (34–50)

B. DESCRIPTION OF BUILDING:

TYPE CONSTRUCTION: I(1), II(2), III(3), IV(4), V(5) [ ] (51) FRAME: STEEL (1), CONCRETE (2), WOOD (3), BEARING WALL (4), OTHER (5) [ ] (52)

STORIES: NUMBER [ ][ ] (53 54) BASEMENT: NO (1), YES (2) [ ] (55)

C. CONSTRUCTION: CONCRETE (1); PRECAST CONCRETE (2); MASONRY (3); STEEL (4); STEEL DECK (5); WOOD (6); OTHER (7)

EXTERIOR WALLS [ ] (56), ROOF [ ] (57), FLOORS [ ] (58), INTERIOR WALLS [ ] (59), PARTITIONS [ ] (60), STAIRS [ ] (61). FALLING HAZARDS: PARAPET (1); CHIMNEY (2); ORNAMENTATION (3); LIGHT FIXTURES (4); WATER TANK (5); CEILINGS (6) [ ] (62)

OCCUPANCY: OFFICE (1); HOTEL (2); APT. (3); WHSE. (4); COMM. (5); SCHOOL (6); HOSPITAL (7); PUBLIC (8); OTHER (9) [ ] (63).

D. DAMAGE NOTED: NONE (1); SLIGHT (2); MODERATE (3); SEVERE (4); TOTAL (5).

EXTERIOR WALLS [ ] (64), FRAME GENERAL [ ] (65), FRAME MEMBERS [ ] (66), FRAME CONNECTIONS [ ] (67), ROOF [ ] (68), FLOORS [ ] (69), INTERIOR WALLS [ ] (70), PARTITIONS [ ] (71), STAIRS [ ] (72), FALLING HAZARDS [ ] (73), MECH. EQUIP. [ ] (74), ELEVATORS [ ] (75), GLASS [ ] (76), PLUMBING [ ] (77), ELECTRICAL [ ] (78).

EXHIBIT 15-4 (2 of 4)

EMERGENCY EARTHQUAKE DAMAGE INSPECTION FORM

E. BUILDING SERIAL NO. ☐☐☐☐ (1–4) (CARD NO. 2)

F. DEGREE OF DAMAGE: MINOR - NO HAZARD (1); DAMAGED (2); MAJOR HAZARD (3); SEVERE HAZARD (4) ☐ (5). SAFETY JEOPARDIZED BY UNSAFE ADJACENT BLDG. OR FALLING HAZARD, NO (1); YES (2) ☐ (6) POSTED: NOT POSTED (1); POSTED GREEN (2); POSTED YELLOW (3); POSTED RED (4) ☐ (7).

VICTIMS MAY BE IN STRUCTURE: NO (1); YES (2); MAYBE (3) ☐ (8).

G. DOLLAR ESTIMATE OF DAMAGE: BUILDING AREA (1000 SF) ☐☐☐☐ (9–12)

ESTIMATED % DAMAGE ☐☐ (13–14) %. ESTIMATED BUILDING VALUATION ($1000) $ ☐☐☐☐☐ (15–19). ESTIMATED VALUATION OF DAMAGE ($1000) ☐☐☐☐☐ (20–24).

H. RECOMMENDATIONS: SHORING & BRACING: NOT NEEDED (1); NEEDED TO PROTECT BUILDING (2); NEEDED TO PROTECT ADJACENT BUILDING (3) ☐ (25). REINSPECTION RECOMMENDED NO (1); YES (2) ☐ (26).

I. SOIL & GEOLOGIC PROBLEMS: SETTLEMENT (1); LIQUEFACTION (2); LANDSLIDE (3); FAULTING (4); OTHER (5) ☐ (27).

J. PHOTOGRAPHS: NOT TAKEN (1); TAKEN (2) ☐ (28).

K. SUMMARY OF DAMAGE:

29 _ 55

56 _ 80

EXHIBIT 15-4 (3 of 4)

EMERGENCY INSPECTION - SUPPLEMENTAL FORM

Structure: By: ______________________

Date Examined: __________ Time __________ Sheet No. __________ of

	ELEMENT	SHUTOFF YES	SHUTOFF NO				SHUTOFF YES	SHUTOFF NO
ELECTRICAL SERVICES	Power Line				MECHANICAL SERVICES	Water Service (if accessible)		
	Substation (if Present)					Sewer Service (if accessible)		
	Main Switchboard and Breakers					Sewage Ejector & Pumps		
	Emergency Generator Transfer Switch (if present)					Fire Pumps Associated Equipment Fuel Storage Tanks Water Storage Tanks		
						Piping & Valves Fuel Gases Medical & Welding Gases Low Flash Point Petroleum Products Standpipes Automatic Sprinkler Systems		

EXHIBIT 15-4 (4 of 4)

REINSPECTION FORM
POST EARTHQUAKE SURVEY
INVESTIGATION AND REPORT SUMMARY

NO. ________

DATE ______________ TIME ______________ INSPECTOR ______________________________

A. Building Location   Code

1. Address ____
2. Street Intersection ____________________________________
3. Side of Street N S E W
4. Occupancy Use ____
5. Owner ______________________________ Telephone ____________

B. Description of Building

1. Type of Construction: I ___; II ___; III ___; IV ___; V ___.
2. Stories: Number ______________
3. Basement: No. ___; Yes ___.
4. Exterior Walls: Describe ________________________________

C. Hazard

Hazard		Posted	Date
1. Safe for occupancy-undamaged or minor damage	________	________	________
2. Limited daytime use-damaged	________	________	________
3. Not to be occupied-hazardous	________	________	________
4. Not to be entered-extreme hazard	________	________	________
5. Not to be entered, hazard from adjacent building	________	________	________

D. Falling Hazards

1. No ___; 2. Yes ___; 3. Yes, adjacent building ___; 4. Describe ____________
____
5. Judged unsafe for surrounding buildings within radius of ______________

E. Shoring

1. Not needed ___; 2. Needed to protect adjacent building ___;
3. Needed to protect building ___.

F. Fire Hazard in Present Condition 1. No. ___; 2. Yes ___

G. Victims Trapped in Building 1. No. ___; 2. Maybe ___; 3. Yes ___; 4. No. ____

H. Summary of Structural Damage ____
____
____

I. Summary of Non-Structural Damage ____________________________________
____
____
____
____

EXHIBIT 15-5 (1 of 1)

REINSPECTION FORM

Structure: By: ____________________

Date Examined: Time: Sheet No. of

STRUCTURAL ELEMENTS
WOOD DECK (__ FLOOR OF __) (ROOF) SYSTEM

ELEMENT	CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
		MAJOR	MINOR
Board or sheet nailing			
Connections to interior bracing elements			
Connections to exterior wall			
Connections to collectors			
Chords			
Joists			
Parapets			
Roof tile			
Other			

EXHIBIT 15-6 (1 of 17)

REINSPECTION FORM

Structure: By: ______

Date Examined: ______ Time ______ Sheet No. ______ of ______

STRUCTURAL ELEMENTS WALLS (___ STORY OF ___ STORIES) WOOD SHEAR WALLS (INTERIOR) (EXTERIOR)

ELEMENT		CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
Board or sheet nailing				
Edge nailing				
Sills	Bolts			
Sills	Grout			
Tiedowns				
Chords at Openings				
Studs				
Wall plumb				
Other				

EXHIBIT 15-6 (2 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________

Date examined __________ Time ________ Sheet No. __________ of __________

STRUCTURAL ELEMENTS
GYPSUM DECK (ROOF) SYSTEM

ELEMENT	CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
		MAJOR	MINOR
Gypsum fill			
Sub-purlins			
Purlins			
Connections to interior bracing elements or shear walls			
Connections to exterior walls or bracing elements			
Connections to collectors			
Chords			
Parapets			
Other			

EXHIBIT 15-6 (3 of 17)

REINSPECTION FORM
Structure: ______________________ By: ______________________________
Date examined: ________ Time ______ Sheet No. __________ of ___________

STEEL DECK (___ FLOOR OF ___) (ROOF) SYSTEM
STRUCTURAL ELEMENTS

ELEMENT		CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
Concrete Fill	Shear or diagonal tension			
Concrete Fill	Connection to steel deck			
Steel Deck	Sheets			
Steel Deck	Welding to bearings			
Steel Deck	Side seams welding or button punching			
Connections to collectors				
Chords				
Connections to interior bracing elements or shear walls				
Connections to exterior bracing elements or shear walls				
Parapets				
Other				

EXHIBIT 15-6 (4 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________________

Date examined: ________ Time: _____ Sheet No. ___________ of __________

STRUCTURAL ELEMENTS (INTERIOR) (EXTERIOR) WALLS

MASONRY WALLS (___ STORY OF ___ STORIES)

ELEMENT		CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
Spandrels over openings	Flexure			
	Shear or diagonal tension			
Piers between openings	Flexure			
	Shear or diagonal tension			
Mortar joints				
Connections to floors (roof)				
Connections at footings				
Wall plumb				
Other				

EXHIBIT 15-6 (5 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________________

Date examined: ________ Time: _____ Sheet No. __________ of __________

(INTERIOR) (EXTERIOR) WALL STRUCTURAL ELEMENTS (___ STORY OF ___ STORIES) MASONRY OR CERAMIC VENEER

ELEMENT		CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
Spandrels over openings	Flexure			
	Shear or diagonal tension			
Piers between openings	Flexure			
	Shear or diagonal tension			
Anchors to frames				
Relieving angles				
Veneer anchors or ties				
Mortar backing				
Mortar joints				
Other				

EXHIBIT 15-6 (6 of 17)

REINSPECTION FORM

Structure: ____________________ By: ____________________

Date examined: ________ Time: ______ Sheet No. __________ of __________

<table>
<tr><th colspan="3">ELEMENT</th><th rowspan="2">CONDITION, OR DESCRIPTION OF DAMAGE</th><th colspan="2">DAMAGE RATING</th></tr>
<tr><th></th><th></th><th></th><th>MAJOR</th><th>MINOR</th></tr>
<tr><td rowspan="7">STRUCTURAL ELEMENTS (__ STORY OF __ STORIES) CONCRETE CAST IN PLACE (INTERIOR) (EXTERIOR) WALLS</td><td rowspan="2">Spandrels over openings</td><td>Flexure</td><td></td><td></td><td></td></tr>
<tr><td>Shear or diagonal tension</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Piers between openings</td><td>Flexure</td><td></td><td></td><td></td></tr>
<tr><td>Shear or diagonal tension</td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Pour joints</td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Plumb</td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Other</td><td></td><td></td><td></td></tr>
</table>

EXHIBIT 15-6 (7 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________________

Date examined: ________ Time: _____ Sheet No. ___________ of ___________

(INTERIOR) (EXTERIOR) WALLS STRUCTURAL ELEMENTS (__ STORY OF __ STORIES) CONCRETE-PRECAST

ELEMENT		CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
Spandrels over openings	Flexure			
	Shear or diagonal tension			
Piers between openings	Flexure			
	Shear or diagonal tension			
Connections of panels to frame				
Connections of panels to each other				
Plumb				
Other				

EXHIBIT 15-6 (8 of 17)

15 Cont.

REINSPECTION FORM

Structure: ______________________ By: ______________________________

Date examined: __________ Time: ______ Sheet No. __________ of __________

CONCRETE - CAST-IN-PLACE STRUCTURAL ELEMENTS (___ FLOOR OF ___) (ROOF) SYSTEM

ELEMENT	CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
		MAJOR	MINOR
Slab			
Joists or waffles			
Beams			
Connections to interior bracing elements or walls			
Connections to exterior walls or bracing elements			
Collectors			
Chords			
Parapets			
Roof tile			
Other			

EXHIBIT 15-6 (9 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________________

Date examined: ________ Time: _____ Sheet No. __________ of __________

CONCRETE - PRECAST STRUCTURAL ELEMENTS (FLOOR OF) (ROOF) SYSTEM

ELEMENT	CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
		MAJOR	MINOR
Concrete fill			
Precast elements			
Precast elements inter-connection			
Connections to interior bracing elements or walls			
Connections to exterior bracing elements or walls			
Connections to collectors			
Chords			
Parapets			
Roof tile			
Other			

EXHIBIT 15-6 (10 of 17)

REINSPECTION FORM

Structure: ______________________ By ______________________________

Date examined: __________ Time: ______ Sheet No. __________ of __________

STRUCTURAL ELEMENTS

HORIZONTAL SYSTEM-SUPPORTING DIAPHRAGM (___ FLOOR OF ___) (ROOF)

(Cast-in-Place Concrete) (Steel) (or Wood) Load Carrying System		ELEMENT	CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
				MAJOR	MINOR
	Beams	Connections to columns			
	Beams	Connections to walls			
	Beams	Flexure or shear			
	Beams	Other			
	Trusses	Connections to columns			
	Trusses	Connections to walls			
	Trusses	Webs			
	Trusses	Chords			
	Trusses	Vertical bracing between trusses			
	Trusses	Other			

EXHIBIT 15-6 (11 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________

Date examined: __________ Time: ______ Sheet No. ____________ of ____________

STRUCTURAL ELEMENTS

HORIZONTAL SYSTEM-SUPPORTING DIAPHRAGM (___ FLOOR ___) (ROOF)

ELEMENT			CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
				MAJOR	MINOR
(Cast-in-Place Concrete) (Steel) (or Wood) Load Carrying System	Lateral Brace System	Connections to columns			
		Connections to walls			
		Webs			
		Chords			
		Vertical bracing between trusses, etc.			
		Wall struts			
		Other			

EXHIBIT 15-6 (12 of 17)

REINSPECTION FORM
Structure: ______________________ By: ______________________________
Date examined: _________ Time: ______ Sheet No. ___________ of ___________

STRUCTURAL ELEMENTS
HORIZONTAL SYSTEM-SUPPORTING DIAPHRAGM (__ FLOOR __) (ROOF)

ELEMENT			CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
				MAJOR	MINOR
Concrete Precast Load Carrying System	Beams	Connections to columns			
		Connections to walls			
		Connections to diaphragm			
		Flexure			
		Shear			
	Trusses	Connections to columns			
		Connections to walls			
		Connections to diaphragm			
		Webs			
		Chords			
		Vertical bracing between trusses			
		Other			

EXHIBIT 15-6 (13 of 17)

15 Cont.

REINSPECTION FORM

Structure:____________________ By: ____________________

Date examined: ________ Time: _____ Sheet No. _______ of _______

ELEMENT				CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
					MAJOR	MINOR
STRUCTURAL ELEMENTS	SUPPORTING FRAME SYSTEM (__ STORY OF __ STORIES) EXTERIOR-INTERIOR	Columns & Beams - Diagonal Bracing	Horizontal or Diagonal Members	Axial compression or concrete		
				Stirrups or ties		
				Main reinforcing		
				Flexure		
				Shear or diagonal tension		
				Other		
			Vertical Members	Axial compression of concrete		
				Stirrups or ties		
				Main reinforcing		
				Flexure		
				Shear or diagonal tension		
				Plumb		
				Other		

EXHIBIT 15-6 (14 of 17)

REINSPECTION FORM

Structure: ______________________________ By: ____

Date examined: ___________ Time: ______ Sheet No. ___________ of ___________

STRUCTURAL ELEMENTS

SUPPORTING FRAME SYSTEM (__ STORY OF __ STORIES) EXTERIOR - INTERIOR

ELEMENT				CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
					MAJOR	MINOR
CAST-IN-PLACE CONCRETE FRAME	Columns & Trusses or Beams - Moment Frame	Horizontal Members	Axial compression of concrete			
			Stirrups or ties			
			Main reinforcing			
			Flexure			
			Shear or diagonal tension			
			Other			
		Vertical Members	Axial compression of concrete			
			Stirrups or ties			
			Main reinforcing			
			Flexure			
			Shear or diagonal tension			
			Plumb			
			Other			

EXHIBIT 15-6 (15 of 17)

REINSPECTION FORM

Structure: ______________________ By: ______________________

Date examined: ________ Time: _____ Sheet No. ________ of ________

SUPPORTING FRAME SYSTEM (___ STORY OF ___ STORIES) EXTERIOR-INTERIOR

STRUCTURAL ELEMENTS

ELEMENT					CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
						MAJOR	MINOR
STEEL FRAME	Columns & Beams - Diagonal Bracing	Horizontal or Diagonal Members	Connection at ends	Flanges			
				Web			
			Member	Bolts			
				Welds			
			Gussetts				
			Other				
		Vertical Members	Member	Flanges			
				Web			
			Anchor bolts				
			Base plate				
			Grout				
			Pilaster at base				
			Plumb				
			Other				

EXHIBIT 15-6 (16 of 17)

REINSPECTION FORM
Structure: ______________________ By: ______________________
Date examined: __________ Time: ______ Sheet No. __________ of __________

SUPPORTING FRAME SYSTEM

STRUCTURAL ELEMENTS (___ STORY OF ___ STORIES) EXTERIOR - INTERIOR

ELEMENT						CONDITION, OR DESCRIPTION OF DAMAGE	DAMAGE RATING	
							MAJOR	MINOR
STEEL FRAME	Columns & Trusses or Beams - Moment Frame	Horizontal or Diagonal Members	Member	Flanges				
				Web				
			End Connection	Bolts				
				Welds				
				Stiffeners				
			Other					
		Vertical Members	Member	Flanges				
				Web				
				Stiffeners				
			Anchor bolts					
			Base plates					
			Grout					
			Pilaster at base					
			Plumb					
			Other					

EXHIBIT 15-6 (17 of 17)

REINSPECTION FORM

Structure: ______________________________ By: ____

Date Examined: ___________ Time: ________ Sheet No. ______________ of _______________

MECHANICAL SERVICES

ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING		SHUTOFF	
		MAJOR	MINOR	YES	NO
Water Service Domestic Fire					
Sewer Service					
Sewage Ejector & Pumps					
Fire Pumps & Associated Equipment Fuel Storage Tanks Water Storage Tanks					
Piping & Valves Fuel Gases Medical & Welding Gases Low Flash Point Petroleum Products Standpipes Automatic Sprinkler Systems					
Boiler & Associated Equipment					
Diesel Generator					
Chiller & Associated Equipment (if essential) Water Pumps Cooling Tower					

EXHIBIT 15-7 (1 of 2)

REINSPECTION FORM
Structure: ______________________ By: ______________________
Date Examined: __________ Time: _______ Sheet No. __________ of __________

MECHANICAL SERVICES

ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING		SHUTOFF	
		MAJOR	MINOR	YES	NO
Storage Tanks Fuel Oil Water Gases Acids					
Oxygen Storage					
Air Compressors					
Fans					

EXHIBIT 15-7 (2 of 2)

REINSPECTION FORM

Structure: ______________________ By: ______________________

Date Examined: ____________ Time: ________ Sheet No. ____________ of ____________

	ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING		SHUTOFF	
			MAJOR	MINOR	YES	NO
ELECTRICAL SERVICES	Power Line					
	Substation (Owner owned)					
	Switchgear (Owner owned)					
	Main Switchboard and Breakers					
	Emergency Generator and Standby Batteries (if present)					
	Transfer Switches and Controls					
	Emergency Distribution Boards					
	Emergency Lighting Panel Board					
	Distribution Board					

EXHIBIT 15-B (1 of 2)

REINSPECTION FORM

Structure: ______________________ By: ______________________

Date Examined: __________ Time: ______ Sheet No. __________ of __________

	ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING		SHUTOFF	
			MAJOR	MINOR	YES	NO
ELECTRICAL SERVICES	Step Down Transformers					
	Motor Control Centers					
	Lighting & Power Panel Board					
	Control Relays & Contactors					
	Lighting Fixtures, Hangers, etc.					
	Fire Alarm System					
	Wiring & Supports					
	Ground Connections					
	Special Systems Equipment					

EXHIBIT 15-8 (2 of 2)

REINSPECTION FORM
Structure: ______________________________ By; ______________________________
Date Examined: ____________ Time: ________ Sheet No. ______________ of ______________

	ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING	
			MAJOR	MINOR
ARCHITECTURAL FINISHES	Exterior Veneer			
	Parapets and Ornaments			
	Ceilings Tee Bar Luminous Gypboard Plaster Other			
	Partitions Tile Plaster Gypboard Gypsum Metal Glass Other			
	Windows			
	Doors			
	Lighting Fixtures			

EXHIBIT 15-9 (1 of 2)

REINSPECTION FORM
Structure: ______________ By: ______________
Date Examined: ________ Time: ______ Sheet No. ________ of ________

ARCHITECTURAL FINISHES

ELEMENT	DESCRIPTION OF DAMAGE	DAMAGE RATING	
		MAJOR	MINOR
Ventilation Fixtures			
Wall Hung Cabinets			
Free Standing Racks & Cabinets			
Steel Stairways			

EXHIBIT 15-9 (2 of 2)

15 Cont.

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COMMENTARY

CHAPTER 1: ADMINISTRATION

Chapter 1 provides general requirements for applying the analysis and design provisions contained in Chapters 3 through 12. It also establishes the mechanism for incorporating a program of systematic abatement of hazards in existing buildings as contained in Chapter 13. The basic character of the Chapter is similar to what might be incorporated in a code as administrative regulations.

The Chapter has been designed so that its use would correlate and be as compatible as possible with normal code administrative provisions, especially as exemplified by the three national model codes. In developing the Chapter, it was recognized that it was not to be written as a regulatory mechanism but rather as the guide for the use of the rest of the document. Therefore, the use of the word "shall" in the Chapter is not implied as a legal imperative but simply as the language necessary to ensure fulfillment of all the steps necessary to technically meet a minimum standard of performance.

It is important to note that certain information within the provisions may be altered by authorities in their adoption process. The most obvious example would be in determining which use groups are adopted within the Seismic Hazard Exposure Groups (Sec. 1.4.2). Such decisions might depend on (a) whether the adopting authority felt that a Group III designation was necessary and therefore that the generally more-demanding design requirements for those buildings are necessary and (b) which uses should be considered as part of Group III or, indeed, in any of the groups. However, it should be strongly emphasized that any "tailoring" procedures such as in the example above should be carefully considered only by highly qualified individuals who are fully aware of all the implications of any changes on all affected procedures in the analysis and design sequences of the document.

Throughout the document there are references to decisions and actions which are delegated to unspecified authorities. The material of the document was written to have applicability to many different types of jurisdictions and chains of authority. The attempt has been made to recognize situations where more than technical decision making can be presumed. In fact the document anticipates the necessity of future standards and approval systems being established to accommodate the use of the document for development of a regulatory system. A good example of this is in Section 1.5, "Alternate Materials and Methods of Construction", where the need for well-established criteria and systems of testing and approval are recognized but there generally are no systems in place to fulfill the needs at this time. In some instances, the decision-making mechanism in the provisions is clearly most logically the province of a regulatory agency or authority. In some cases it appears that the authority may be a law-making body such as a legislature or city council, etc. In some cases the decisions may be the province of a policy-making body which may be either local or statewide.

Some attempt has been made to differentiate between the different types of responsible officials without trying to specifically delegate authority. Therefore, the terms "Cognizant Jurisdiction" and "Regulatory Agency" are used. In cases where the provisions are traditionally (model codes, etc.) the responsibility of a Regulatory Agency such as a building official, or department, or similar, the term "Regulatory Agency" is used. Where it is recognized that the provision might involve matters relating to other authorities, the term "Cognizant Jurisdiction" is used. The Cognizant Jurisdiction may very well be the building official, but it may also be a law-making body or a policy-making body or official.

A good example of the need of such generality is provided by the California State Law relative to design and construction of schools. That law establishes requirements for independent special inspection approved and supervised by the Office of the State Architect. This introduces control by an office at a state level which does not exist in many of the states.

Sec. 1.1 PURPOSE

The stated purpose of the provisions is to minimize hazard to life in buildings from earthquakes based on anticipated conditions of shaking. There are also provisions to enable designers to design for the survival of a certain functional capacity level of operations within the building. The bases for establishing the anticipated conditions of shaking are explained more fully in the commentary for Sec. 1.4.1.

Sec. 1.2 SCOPE

The scope statement establishes in general terms the applicability of the provisions as a base of reference. Certain buildings are exempted and need not comply.

1. Buildings for agricultural use are generally excepted by most codes from code requirements because of the exceptionally low risk to life involved.

2. Normal one- and two-family dwellings in Seismic Index Areas 1 and 2 are excepted because they represent exceptionally low risks.

Because of the unique structural character of the special structures identified in this Section, and other structures which are similar in character, it is impossible to provide a single standard of reference which would ensure an adequate identification of response characteristics and methods of design and still be usable by the majority of designers.

Sec. 1.3 APPLICATION OF PROVISIONS

The requirements for the application of the provisions of Chapters 2 through 12 to new and existing buildings are established in this Section.

1.3.1 NEW BUILDINGS

A simple procedure is established for one- and two-story wood frame dwellings in regions of higher seismicity. While some control is necessary to ensure the integrity of such structures it is felt that the requirements of Sec. 9.4 and 9.6 are adequate to provide the safety required based on the history of such frame construction--especially low structures--in earthquakes.

1.3.2 EXISTING BUILDING ALTERATIONS AND REPAIRS

Alterations and repairs to existing buildings may require a building permit, depending on the requirements of the local building regulations being used. The national model codes have similar conditions under which a building permit is required, and generally it can be said that a permit is required when anything except what is defined by the code as "ordinary repairs" is involved. The object of this provision is to ensure that adequate consideration is given to the effects of repairs and alterations on the overall seismic performance characteristics of the structure. The provision says that where applicable to the work being done the requirements of this document should be used. In many cases this will require an analysis of the as-built structure incorporating the effects of proposed changes.

In cases where the structure already exceeds the requirements for seismic force resistance which would be required of a new building of the same Seismic Performance Category (Seismicity Index and Seismic Hazard Exposure Group), alterations and repairs may be made in such a way that the seismic force resistance is reduced to that required of new buildings of the same Seismic Performance Category.

In cases where the building does not exceed the seismic force resistance required of a new building of the same Seismic Performance Category, the alterations and repairs cannot result in a reduction of the existing seismic force resistance of the building.

1.3.3 CHANGE OF USE

When buildings are subject to changes of use, it is possible that the new use may place the building into a different Seismic Performance Category. If a portion of the building is changed in use, then Sec. 1.4.2(D) would apply. If the change of use results in a change of the Seismic Performance Category to a higher category according to Sec. 1.4.3, then the building must be made to conform to the requirement for the new Category.

1.3.4 SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

As more attention is directed toward the possible hazards of existing buildings due to seismic shaking, it is expected that certain local and statewide programs will be instituted by those authorities to systematically abate the hazard to the degree and over the period of time which appears justifiable in terms of both life safety and economic reasonableness. Therefore, extensively detailed Chapters 13 and 14 have been included to provide guidelines for establishing and implementing such programs.

Sec. 1.4 SEISMIC PERFORMANCE

The requirements for analysis and design of buildings in this document are based on a seismic hazard criterion which is the result of the relationship between the use of the building and the level of shaking to which it may be exposed. This relationship reflects primarily the concern for life safety and therefore the degree of exposure of the public to hazard based on the measure of risk.

The purposes of Sec. 1.4.1 and 1.4.2 are to provide the means for establishing a measure of seismic risk for a building of any use group and in any area of the United States. Based on this measure the key to the application of the provisions, including when Quality Assurance procedures are required (Sec. 1.6), is identified. This key is the Seismic Performance Category of Table 1-A.

1.4.1 SEISMICITY INDEX AND DESIGN GROUND MOTIONS

An extensive discussion of this Section is provided at the end of the commentary for this Chapter.

1.4.2 SEISMIC HAZARD EXPOSURE GROUPS

Historically, the typical occupancy classifications in building codes are based on the potential hazards associated with fire. Review and evaluation of existing building code provisions indicated that most occupancy-type classifications do not meet the purpose of this document. For example a large-scale enclosed-mall-type regional shopping complex is a relatively new architectural form representing a potentially high risk occupancy, which existing codes do not specifically address properly. These classifications are based not only on different considerations than those related to seismic resistance, but in some cases on considerations which are contrary to good seismic performance.

Attention was given to the Model Code Standardization Committee (MSCS) code change Proposal III-75-1 which recommended a series of change of occupancy designations so as to refer to the same use in all model codes. However, these MCSC changes did not seem sufficiently varied to cover all issues related to seismic safety, being limited to only seven broad, general, fire-oriented classifications as follows:

Assembly
Business (including offices, factories, mercantile, and storage)
Educational
Hazardous
Institutional
Miscellaneous Structures
Residential

A new approach was needed for defining occupancy exposure to seismic hazards based on a commonality of conditions proposed for the use of a building facility or space. Conditions would involve evaluation of parameters consisting of, but not limited to:

1. The number, age and condition of the persons normally expected to be within or without the immediate environs of the building.

2. The size, height and area of the building(s).

3. The spacing of the buildings to public rights-of-way over which the designer has no control relative to the future number of persons exposed to risk by the buildings.

4. The varying degreee of built-in or brought-in hazards based on possible use of the building.

Accordingly, during the detailed initial phase, occupancy types were regrouped and expanded to cover a complete range of factors critical to seismic safety in terms of life loss. The expanded classification types presented in the "Tentative Matrix", Table C8-5, (see Commentary for Chapter 8) were developed for study purposes only and are not intended as recommended changes to any building code. They were derived from the classifications defined in the Uniform Building Code, 1973 Edition.

In terms of post-earthquake recovery and redevelopment, certain types of occupancies are vital to public needs. These special occupancies were identified and given specific recognition. In terms of disaster preparedness, fire and police stations, hospitals, and regional communication centers, identified as critical emergency services, should not be included in the same classification as retail stores, office buildings, and factories as is presently the case in some codes.

Because of vital public needs immediately following a natural disaster, attention was given to the preservation of strategic contents in distinct building types. For example, should storage facilities for medical supplies, critical foodstuffs, and other emergency materials require a higher seismic performance than the storage of less vital reserves and provisions. It was noted that disaster recovery officials initally considered the identification and protection of critical stocks needed during or immediately following an earthquake to be of paramount importance. This was not to imply that all warehouses and storage facilities must be designed for the ultimate protection of any or all contents. What was indicated was that warehouse facilities should be designed on the basis of their maximum level of intended function or, to state it another way, medical supply warehouses being designed under higher standards may house anything, while storage facilities of lesser ratings may not store critical supplies unless brought up to a higher level of seismic performance.

Subsequent discussions with disaster recovery officials revealed that emergency contingency plans contemplated bringing needed medical and other recovery items including foodstuffs into a disaster area from outside staging areas. Therefore, no separate category of warehousing was required for the storage of critical materials. Table C8-3 thus has ten occupancy groups, A through I, with some individual occupancies and groups bearing little or no relationship to current code groupings.

The occupancies were then consolidated into five basic groups by making a few compromises. This consolidation was done in an effort to place those occupancies initially listed in the Tentative Matrix into groups that shared common components performance criteria. The consolidation indicated that these groups were easily identifiable by use patterns; confirmation of the original occupancy-component-performance criteria rating. This intermediate grouping was:

Group I	Group II	Group III	Group IV	Group V
Fire	Public Assembly	Restrained Occupants	Aircraft Hangars	Private Garages
Police	Open Air Stands	Nurseries (non-ambulatory)	Woodworking	Sheds
Hospitals	Day Care	Ambulatory	Factories	Barns
	Schools		Repair Garages	
	Colleges		Service Stations	
	Retail Stores		Storage Garages	
	Shopping Centers		Wholesale	
	Offices		General Warehouse	
	Hotels		Printing Plants	
	Apartments		Factories	
	Emergency Vehicles		Ice Plants	
	Power Utilities		Dwellings	
			Hazardous Flammable Storage	
			Less Hazardous Flammable Storage	

The final occupancy grouping in Table C8-4 resulted from a logical consolidation of Table C8-3, consideration of code enforcement problems, and the need to use a common hazard exposure grouping for all of the design provisions. It is felt that this grouping can be augmented as local conditions warrant. Specific consideration was given to Group III, essential facilities, to insure that only those facilities specifically designated by the cognizant jurisdiction would be included because this determination has both political and economic impact.

Group II category contains those occupancies that have large numbers of occupants either due to the overall size of the building or the number of stories; the character of the use, such as public assembly, schools, or colleges; or where the height exposes the occupants to greater life safety hazard. Other considerations included uses wherein the occupants were restrained or otherwise handicapped from moving freely, such as day care centers, hospitals, and jails.

Group I contains all uses other than those excepted generally from the provisions in Sec. 1.2. Those in Group I have lesser life hazard only insofar as there is the probability of lesser numbers of occupants in the buildings and that the buildings are lower and/or smaller. The height of four stories was used in part due to the general model code use of this height as being the maximum allowable height for wood frame and masonry/wood framed classes of buildings (designated Types 5 and 3, respectively, in the 1976 Uniform Building Code).

C1.4.2 Cont.

In buildings with multiple uses, the building shall be assigned the classification of the highest Seismic Hazard Exposure Group which occupies 15 percent or more of the total building area. Such assignments should also be considered when changes are made in the use of a building. For example, if a portion subject to change of use is in a building of Seismic Hazard Exposure Group I, and the portion represents 15 percent or more of the total building area and is a use found in Seismic Hazard Group II, then the entire building should be reclassified to Group II, and the appropriate Seismic Performance Category applies based on the appropriate Seismicity Index and the Seismic Hazard Exposure Group II classification.

Consideration was given to possibly reducing the number of groupings by combining Groups I and II and leaving Group III the same as it is stated herein above. It was the concensus of those involved that such a merging would not be responsive to the relative life hazard problems.

1.4.3 SEISMIC PERFORMANCE CATEGORIES

This Section establishes the four design categories which are the keys for establishing requirements for any building based on its Seismicity Index and use (Seismic Hazard Exposure Group). Once the Seismic Performance Category (A, B, C, or D) for the building is established, many other requirements such as detailing, quality assurance, limitations, specialized requirements, and even applicability of the provisions to alterations and repairs and change of use are related to the Seismic Performance Category.

1.4.4 SITE LIMITATION FOR SEISMIC DESIGN PERFORMANCE CATEGORY D

Essential facilities which may be required after an earthquake and are located in zones of higher seismicity should not be located over an active fault. Although some structures could and may be designed to remain intact even if a fault occurs at the base, knowingly exposing an essential facility to such a risk is unreasonable and should be unnecessary.

Sec. 1.5 <u>ALTERNATE MATERIALS AND METHODS OF CONSTRUCTION</u>

It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction either existing or anticipated. While not citing specific materials or methods of construction currently available which require approval, this Section serves to emphasize the fact that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgement of the best use of the materials and methods based on well-established expertise. It is important that any replacement or substitute be evaluated on a basis of an understanding of all the ramifications of performance, strength, and durability implied by the provisions.

At the same time it is recognized that until other approval standards and agencies are created, regulatory agencies will have to operate on the basis of the best evidence available to substantiate any application for alternates. It is strongly recommended where there is an absence of accepted standards that applications be supported by extensive reliable data obtained from tests simulating, as closely as is practically feasible, the actual load and/or deformation conditions to which the material is expected to be subjected during the service life of the building. These conditions, where applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

Sec. 1.6 <u>QUALITY ASSURANCE</u>

Building failures during earthquakes which are directly traceable to poor quality control during construction are innumerable. The literature is replete with reports pointing out that collapse may have been prevented had proper inspection been exercised.

The remarkable performance during earthquakes by California schools constructed since 1933 is due in part to the rigorous supervision of construction required by State law. Independent special inspection, approved and supervised by the Office of the State Architect is an important feature of those requirements. Such an excellent record of performance has influenced the writing of these provisions so as to rely heavily on the concept of special inspection to ensure good construction.

Recognizing that there must be coordinated responsibility during construction, these provisions set forth the role each party is expected to play in construction quality control. The building designer specifies the quality assurance requirements, the contractor exercises the control to achieve the desired quality and the owner monitors the construction process through special inspection to protect the public interest in safety of buildings. It is essential that each party recognize its responsibilities, understand the procedures, and be capable of carrying them out. Because the contractor and the specialty subcontractors are doing the work and exercising control on quality, it is essential that the special inspection be performed by someone not in their direct employ and also be approved by the Regulatory Agency. When the owner is also the builder, he should engage independent agencies to conduct these inspections rather than trying to qualify his own employees.

The approach used in preparing these provisions was to borrow liberally from the pattern already established by the ICBO Uniform Building Code (UBC) 1976 Edition which details structural quality provisions under the Administrative portion of that Code, Chapter 3, Section 305, "Special Inspections".

There are two major differences, however, between these tentative provisions and those of the Uniform Building Code. First, these provisions cover only those portions and components of the building that are directly affected by earthquake motions and whose response could affect life safety and continued functioning of the building (where designated). Second, these provisions for the first time attempt to place minimum quality assurance requirements on installation of nonstructural components which are designated as deserving special attention during construction. These are described as "Designated Seismic Systems" throughout and are defined as being "the Seismic Resisting Systems and those architectural, electrical, and mechanical systems and their components which require special performance characteristics". This means that the designer most familiar with the requirements of each system must spell out in a Quality Assurance Plan those components which will require special inspection and tests during construction to assure their ability to perform satisfactorily during earthquakes.

The provisions are concerned with those components which affect the building performance during an earthquake and/or which may be adversely affected by earthquake motions as specified under other sections of the provisions. The requirements under Sec. 1.6 are minimum and it could very well be the decision of the designers to include all phases of construction throughout the project under a quality assurance plan. For many buildings the additional cost to do so would be minimal. The primary method of achieving quality assurance is through the use of specially qualified inspectors approved by the Regulatory Agency. The number of such inspectors actually employed will vary widely depending upon the size and complexity and function of the building. These provisions permit the designer or his employee to perform these inspections as long as they are approved by the Regulatory Agency having jurisdiction and can demonstrate reasonable competence in the particular category of work they inspect.

1.6.1 QUALITY ASSURANCE PLAN

Introduced herein is the concept that the quality assurance plan must be prepared by the person responsible for the design of each seismic system subject to quality assurance whether it be architectural, electrical, mechanical, or structural in nature. The quality assurance plan may be a very simple listing of those elements of each system which have been designated as being important enough to receive special inspection and/or testing. The extent and duration of inspection must be set forth as well as the specific tests and the frequency of testing.

Although some design professionals have expressed reluctance to assume this duty because of an assumed increase in potential liability, it has been demonstrated by performance of schools in California earthquakes that the improved quality also acts to protect the professional. Furthermore, the design professional is the most qualified person to prepare such a plan as he is the most familiar with the design concept.

The Regulatory Agency, however, must approve the quality assurance plan and must obtain from each responsible contractor a written statement that he understands the requirements of the quality assurance plan and that he will exercise control to obtain conformance. The exact methods of control are left up to the individual contractor subject to approval by-the Regulatory Agency. However, special inspection of the work is required in specific situations to give the agency reasonable assurance that the approved drawings and specifications are followed.

1.6.2 SPECIAL INSPECTION

The requirements listed in this Section from Foundations through Structural Wood are basically the same as those currently requiring special inspection under the 1976 UBC and it is a premise of the provisions that there will be available an adequate supply of knowledgeable and experienced inspectors to draw upon for the structural categories of work. Special training programs may have to be developed and implemented for the nonstructural categories.

A Special Inspector is defined as a "specially qualified person approved by the Regulatory Agency-to perform special inspection". As a guide to such agencies, it is contemplated that he may be one of the following:

1. A person employed by and supervised by the design architect or engineer of record who is responsible for the design of the designated seismic system for which the Special Inspector is engaged.

2. A person employed by an approved inspection and testing agency who is under the direct supervision of a registered engineer also employed by the same agency.

3. A manufacturer or fabricator of components, equipment, or machinery who has been approved for manufacturing components meeting seismic safety standards and who maintains a quality control plan approved by the Regulatory Agency. Evidence of such approval must be clearly marked on each designated seimsic system component shipped to the jobsite.

Sec. 1.6.2(H). It is anticipated that the minimum requirements for Architectural Components will be complied with when the Special Inspector is satisfied that the method of anchorage or fastening and the number, spacing, and types of fasteners actually used conform with the plans and specifications for the component installed. It is noted that such special inspection requirements are only for those components with Superior (S) or Good (G) performance as required in Chapter 8 and then only in areas having a Seismicity Index of 3 or 4.

Sec. 1.6.2(I). In addition to verification of the fastening and anchorage for mechanical and electrical components, it is anticipated that the Special Inspector will verify that the designated components are labeled to meet S or G performance standards as required in Chapter 8 and as established by the Regulatory Agency.

C1.6.2 Cont.

In the initial applications of this Section close cooperation between the designer, manufacturer, Special Inspector, and Regulatory Agency must be exercised until all learn their respective roles and a definite inspection routine is established.

1.6.3 SPECIAL TESTING

The specified testing of the structural materials follows procedures and tests long established by industry standards. A possible exception is masonry where there is presently no single nationally accepted standard which encompasses all of the diversity of materials now being used in masonry construction. The acceptance criteria should be agreed upon prior to contract award.

1.6.4 REPORTING AND COMPLIANCE PROCEDURES

The success of a quality assurance plan depends upon the intelligence and knowledge of the inspector and the accuracy and thoroughness of his reports. It should be emphasized that both the Special Inspector and the contractor are required to submit to the Regulatory Agency a final certification as to the adequacy of the completed work. The contractor, with his day-to-day knowledge of the installation, is in the best position to state whether or not all the construction has been completed in accordance with approved plans and specifications. To be fully aware, however, the contractor must institute a system of reporting within his own organization which enables him to effectively practice quality control. The inspector can only attest to the work he has personally inspected, and therefore acts more as an auditor or monitor of the quality control program exercised by the contractor.

1.6.5 APPROVED MANUFACTURERS CERTIFICATION

Provision is made for the special approval of manufactured designated components. This arises because most of the mechanical or electrical equipment is manufactured off-site and is delivered to the job in its own container. The Special Inspector being at the jobsite cannot judge the adequacy of anchorage or the seismic resistance of the equipment contained therein and in most instances cannot be present during the off-site manufacturing. It is expected therefore that a system of approvals and labeling must be established by the Regulatory Agency in much the same way as labeling of firedoors is presently being done.

SEC. 1.4.1 SEISMICITY INDEX AND DESIGN GROUND MOTIONS. DETERMINATION OF A_a AND A_v COEFFICIENTS AND DEFINITIONS OF SEISMICITY INDEX

A. INTRODUCTION

This portion of the commentary gives the background for the two ground motion regionalization maps in Sec 1.4.1 and the seismic design coefficient, C_s, in Sec. 4.2.

It must be emphasized at the outset that the specification of earthquake ground shaking for design cannot be achieved solely by following an agreed upon set of scientific principles. First, the causes of earthquakes are still poorly understood and experts do not agree how the knowledge which is available should be interpreted to specify ground motions for use in design. Second, to achieve workable building code provisions it is necessary to simplify greatly the enormously complex matter of earthquake occurrence and ground motions. Finally, any specifications of a design ground shaking implies a balancing of the risk of that motion occurring against the cost to society of requiring that structures be designed to withstand that motion. Hence judgement, engineering experience, and political wisdom are as necessary as science. In addition, it must be remembered that the design ground shaking does not by itself determine how a structure will perform during a future earthquake; there must be a balance between the specified shaking and the rules used to translate that shaking into a design.

The recommended regionalization maps and seismic design coefficients are the result of a collective judgement by several committees, based upon the best scientific knowledge available in 1976, adjusted and tempered by experience and judgment. The following sections strive to explain the bases for the various recommendations, as a guide both to the user of the provisions and to those who will improve the provisions in the future. It is expected that the maps and coefficients will change with time, as the profession gains more knowledge about earthquakes and their resulting ground motions and as society gains greater insight into the process of establishing acceptable risk.

B. POLICY DECISIONS

The recommended ground shaking regionalizations maps are based upon several policy decisions, the first two of which are departures from past practice in the United States.

The first decision was that the relationship should take into account the distance from anticipated earthquake sources. This decision reflects the observation that the higher frequencies in ground motion attenuate more rapidly with distance than the lower frequencies. Thus, at distances of 100 km or more from a major earthquake, flexible buildings may be more seriously affected than stiff buildings. To accomplish the objective of this policy decision, it proved necessary to use two separate ground motion parameters and therefore to prepare two maps.

The second policy decision was that the probability of exceeding the design ground shaking should--as a goal--be roughly the same in all parts of the country. This contrasts to the zoning maps currently in use in the United States, which have been based upon estimates of the maximum ground shaking experienced during the recorded historical period without consideration of how frequently such motions might occur. There is not unanimous agreement in the profession with this policy decision. In part, this lack of agreement reflects doubt as to how well the probability of ground motion occurrence can be estimated with today's knowledge and disagreement with the specific procedures used to make the estimates, rather than any true disagreement with the goal. Further, it really is the probability of structural failures with resultant casualties that is of concern, and the geographical distribution of that probability is not necessarily the same as the distribution of the probability of exceeding some ground motion. (This point is discussed further in Section L. Sections A through M are part of this Commentary.) Thus the goal as stated is not necessarily the ideal goal, but is judged to be the most workable goal for the present time.

The second policy decision implies that the design ground shaking is not necessarily the most intense motion that might conceivably occur at a location. This is not a new policy decision; this policy is implied by past codes. It does seem well to state the matter very clearly: It is possible that the design earthquake ground shaking might be exceeded during the lifespan of the structure - although the probability of this happening is quite small. In this connection, several points must be emphasized. First, considering the significant cost of designing a structure for extreme ground motions, it is undesirable to require such a design unless there is a significant probability that the extreme motion will occur or unless there is a particularly severe penalty associated with failure or nonfunctioning of the structure. Second, a building properly designed for a particular ground motion will provide considerable protection to the lives of occupants during a more severe ground motion. Third, even if it were desirable to design for the extreme ground motion (or maximum credible motion--various names have been suggested), it is virtually impossible, at this time, to get agreement as to how intense this motion might be. This is especially true for the less seismic portions of the country.

There was a third important policy decision, which also is not a new policy: the regionalization maps should not attempt to microzone. In particular, there was to be no attempt to locate actual faults on the regionalization maps, and variations of ground shaking over short distances--on a scale of about 10 miles or less--were not to be considered. Any such microzoning must be done by experts who are familiar with localized conditions. There are many local jurisdictions which should undertake microzoning; this point is discussed further in Sections E and J.

C. DESIGN EARTHQUAKE GROUND MOTION

The previous sections have spoken loosely about a "design ground shaking" without being specific as to the meaning of the phrase. A precise definition is very difficult if not impossible, but the concept is straightforward enough. The "design ground shaking" for a location is the ground motion which an architect or engineer should have in mind when designing a building which is to provide protection for life safety.

At the present time, the best workable tool for describing the design ground shaking is a smoothed elastic response spectrum for single degree-of-freedom systems (Newmark and Hall, 1969). Such a spectrum provides a quantitative description of both the intensity and frequency content of a ground motion. Smoothed elastic response spectra for 5 percent damping were used as a basic tool for the development of regionalization maps and for the inclusion of the effects of local ground conditions. In effect, the second policy decision was reinterpreted to mean for all locations roughly equal probability of exceeding at all structural periods the ordinates of the design elastic response spectrum for that location. Again, this statement should be looked upon as a general goal, and not one that can be strictly met on the basis of present knowledge.

This does not mean that a building must necessarily be designed for the forces implied by an elastic response spectrum. Sections H and I describe how, for purposes of the proposed design provisions, elastic response spectra were converted into a formula for seismic design coefficient. For structures which can safely strain past their yield point, the forces determined in accordance with Sec. 4.2 are significantly smaller than those which would be determined from the corresponding elastic spectrum. However, the designing engineer would do well to keep the probable design ground motion in mind.

A smoothed elastic response spectrum is not necessarily the ideal means for describing the design ground shaking. It might be better to use a set of four or more acceleration time histories whose average elastic response spectrum is similar to the design spectrum. This approach may be desirable for buildings of special importance but is not feasible for the vast majority of buildings. The use of a single time history generally is not adequate. This emphasizes that the design ground shaking is not a single motion, but rather a concept that encompasses a family of motions having the same overall intensity and frequency content but differing in some potentially important details of the time sequences of motions.

A significant deficiency of the response spectrum is that it does not by itself say anything about the duration of the shaking. To the extent that duration effects elastic response, it is accounted for by the spectrum. However, the major effect of duration is upon possible loss of strength once a structure yields. Duration effects have not been considered explicitly in drawing up the recommended provisions, although in a general way it was envisioned that the design ground shaking might have a duration of 20 to 30 seconds. The possibility that the design motion might be longer in highly seismic areas and shorter in less seismic areas was one of the considerations which influenced the assignment of Seismicity Index values in Sec. 1.4.

D. GROUND MOTION PARAMETERS

In developing the design provisions, two parameters were used to characterize the intensity of design ground shaking. These parameters are called the Effective Peak Acceleration (EPA), A_a, and the Effective Peak Velocity (EPV), A_v. These parameters do not at present have precise definitions in physical terms but their significance may be understood from the following paragraphs.

To best understand the meaning of EPA and EPV, they should be considered as normalizing factors for construction of smoothed elastic response spectra (Newmark and Hall, 1969) for ground motions of normal duration. The EPA is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 seconds, while the EPV is proportional to spectral ordinates at a period of about 1 second (McGuire, 1975). The constant of proportionality (for a 5 percent damping spectrum) is set at a standard value of 2.5 in both cases.

For a specific actual ground motion of normal duration, EPA and EPV can be determined as illustrated in Figure C1-1. The 5 percent damped spectrum for the actual motion is drawn, and fitted by straight lines between the periods mentioned above. The ordinates of the smoothed spectrum are then divided by 2.5 to obtain EPA and EPV. The EPA and EPV thus obtained are related to peak ground acceleration and peak ground velocity but are not necessarily the same as or even proportional to peak acceleration and velocity. When very high frequencies are present in the ground motion, the EPA may be significantly less than the peak acceleration. This is consistent with the observation that chopping off the highest peak in an acceleration time history has very little effect upon the response spectrum computed from that motion, except at periods much shorter than those of interest in ordinary building practice. Furthermore, a rigid foundation tends to screen out very high frequencies in the free field motion. On the other hand, the EPV will generally be greater than the peak velocity at large distances from a major earthquake (McGuire, 1975). Ground motions increase in duration and become more periodic with distance. These factors will tend to produce proportionally larger increases in that portion of the response spectrum represented by the EPV.

If an earthquake is of very short or very long duration, then it is necessary to correct the EPA and EPV values to more closely represent the event. It is well documented that two motions having different durations but similar response spectra cause different degrees of damage--the damage being less for the shorter duration. In particular, there have been numerous instances where motions with very large accelerations and short durations have caused very little or even no damage. Thus, when expressing the significance of a ground motion to design, it is appropriate to decrease the EPA and EPV obtained from the elastic spectrum for a motion of short duration. On the other hand, for a motion of very long duration it would be appropriate to increase the EPA and EPV. There are at present, however, no agreed-upon procedures for determining the appropriate correction; it must be done by judgement.

Thus the EPA and EPV for a motion may be either greater or smaller than the peak acceleration and velocity, although generally the EPA will be smaller than peak acceleration while the EPV will be larger than the peak velocity. Despite the lack of precise definitions, the EPA and EPV are valuable tools for taking into consideration the important factors relating ground shaking to the performance of a building.

At any specific location either the EPA or the EPV may govern the design of a building. In general, however, it is desirable to know both values.

For purposes of computing the lateral force coefficent in Sec. 4.2, EPA and EPV are replaced by dimensionless coefficients A_a and A_v respectively. A_a is numerically equal to EPA when EPA is expressed as a decimal fraction of the acceleration of gravity; e.g., if EPA = 0.2g, then $A_a = 0.2$. A_v is proportional to EPV, as explained in Sec. L.

E. MAP FOR EFFECTIVE PEAK ACCELERATION

The development of a map for EPA for the contiguous 48 states was facilitated by the work of Algermissen and Perkins (1976). Their map, which is reproduced here as Figure C1-2, is based upon the principles of seismic risk (Cornell, 1968; Algermissen and Perkins, 1972). Several steps are involved in the preparation of such a map:

1. Source zones and faults, in which or along which significant earthquakes can occur, are identified and brought together on a source zone map.

2. For each source zone or fault the rate at which earthquakes of different magnitude can occur, and the maximum credible magnitude are estimated.

3. Attenuation laws are used to give the intensity of shaking as a function of magnitude and distance from an epicenter.

4. With the foregoing information as input, a computer program based on probabilistic principles can generate values which are then used to produce contours of locations with equal probabilities of receiving specific intensities of ground shaking.

Algermissen and Perkins relied primarily upon historical seismicity, adjusted where possible by geological and tectonic information. The Algermissen-Perkins map shows contours of peak acceleration on rock which have a 10 percent probability of being exceeded in 50 years.

A contour map for EPA for the contiguous states was developed by the ATC-3 study, and is given in Figure C1-3. (This map was later converted into the map in Figure 1-1 of Chapter 1 by shifting contours to lie along county lines; see Section J.) It gives EPA for firm ground, which includes shallow deposits of stiff cohesive soils and dense granular soils as well as rock.

The map of EPA is in many ways quite similar to the Algermissen-Perkins map, and indeed was influenced by preliminary versions of that map. In adapting a map such as the Algermissen-Perkins map to the purposes of these recommended provisions, it was necessary to judge how acceleration as used in their study is related to EPA, and how the "rock" of their study relates to the "firm ground" of the recommended provisions. To produce a map appropriate as a basis for design it is desirable to use smoothed contours, and further it is necessary to decide how to treat an area--such as New England and the Middle Atlantic states--where the accelerations in the Algermissen-Perkins map lie just below one of the arbitrarily selected contour levels. Seismologists from various parts of the country were asked to comment on proposed versions of the EPA map, and suggested what were in effect alternate versions of the source areas. Other proposed maps--prepared from data in Culver et al (1975) and published by Wiggins et al (1977), Foss (1977), and others, using similar principles but different interpretations of historical seismicity and geological evidence--were studied. All of this evidence was taken into account, where deemed appropriate, by adjusting the locations of contours for EPA. Figure C1-3, having literally been drawn by a committee, lacks some of the internal consistency of the Algermissen-Perkins map, but is judged to provide the best current estimate of the geographic variation of EPA for purposes of design.

Perhaps the most significant difference between Figures C1-2 and C1-3 occurs in the area of highest seismicity in California. Within this region, the Algermissen-Perkins map has contours of 0.6g. On the other hand, the map for EPA has no values higher than 0.4g. There are several different reasons for this difference, all of which contributed to the decision to limit EPA to 0.4g. One factor is the basic difference between peak acceleration and EPA. There is doubt among many professionals that large earthquakes really will cause very large accelerations except in quite localized spots influenced by topography. Many also believe that there is an upper limit to the acceleration that can be transmitted even through dense soil. There is also the argument that a building code requiring design for an EPA greater than 0.4g will not really bring about more earthquake-resistant construction. Finally, while by the formal logic used to establish EPA there may be locations-inside of the 0.4g contour where higher values would be appropriate, contouring such small areas would amount to microzoning. In short, the decision to limit the EPA to 0.4g was based in part upon scientific knowledge and in part upon judgment and compromise.

Figure C1-4 presents maps of EPA for Alaska, Hawaii, and Puerto Rico. In these areas no studies of the type such as produced by Algermissen and Perkins were available. However, there have been a number of seismological studies and seismic risk analyses in connection with the Alaskan pipeline, proposed nuclear power plants, etc. There also existed past and proposed seismic zoning maps. All of this information was used to construct maps of EPA that were judged to be consistent with the map for the contiguous 48 states.

It has already been noted that the Algermissen-Perkins map was heavily influenced by historical seismicity; that is, by the pattern of earthquakes that have occurred during the past 150 years (on the west coast) to 350 years (on the east coast). Where there was solid geological evidence that this rather short period of history might be misleading, this evidence was incorporated into the source model. This approach does mean that areas which have not experienced significant earthquakes during the historical period, and for which there is no solid geological basis for suspecting that such earthquakes might occur, end up being designated as areas of low seismic risk. Careful examination of old earthquake records is necessary as some historic events felt in one location and recorded as being centered on that location may actually have been a larger distant event. These same difficulties apply to the map of EPA, although some very recent geological and seismological studies did lead to the EPA being increased in some parts of the country where the historical record alone would indicate low seismicity.

Critics of the seismic risk approach rightfully argue that the historical record is far too short to justify the extrapolations inherent in the approach. Moreover, the most widely used procedures assume that large earthquakes occur randomly in time, so that the fact that a large earthquake has just occurred in an area does not make it less likely that a large earthquake will occur next year. In the light of modern understanding of earthquake occurrences, this assumption is of limited validity. However, at present there is no workable alternative approach to the construction of a seismic design regionalization map which comes close to meeting the goal of the second policy decision.

F. MAP OF EFFECTIVE PEAK VELOCITY

No general mapping study is currently available for EPV. Hence, the maps for EPV (Figures C1-5 and C1-6) were constructed by modifying the map for EPA, using the principles described in the following paragraphs. Since EPV is velocity, it is appropriately expressed in units such as inches per second. For ease in developing the formulas in Sec. 4.2, it proved desirable to also express EPV by a dimensionless parameter (Av) which is an acceleration coefficient. This parameter is referred to as velocity-related acceleration coefficient. Figures C1-5 and C1-6 show contours of A_v. The relationship between EPV and A_v is given by the following table:

Effective Peak Velocity (in./sec)	Velocity-related Acceleration Coefficient, Av
12	0.4
6	0.2
3	0.1
1.5	0.05

The first step was to assume that the elastic response spectrum for firm ground would apply along the contours for EPA = 0.4g in Figure C1-3. The shape of this response spectrum, as described in Section H, was obtained from analyses of actual strong motion records at distances of 20 to 50 miles from moderate to large earthquakes in California. To construct this spectrum, if EPA = 0.4g it is necessary to have EPV = 12 inches per second.

A similar assumption was made for all the peaks of the contour map for EPA; that is, at all locations where a contour gives the highest EPA in a region. For example, the EPV was set at 3 inches per second along the contour for EPA = 0.1g in the vicinity of the Appalachian Mountains and South Carolina.

A study by McGuire (1975), based upon strong motion records in California, has provided data concerning the attenuation of EPV with distance. For an earthquake of large magnitude, it was found that the distance required for EPV to decrease by a factor of 2 is about 80 miles. Thus, in the western part of the country, the contours for EPV = 6 inches per second were located at a distance of about 80 miles outside of the contours for EPV = 12 inches per second. Similarly, in Washington and Utah where the highest contour is at 0.2g, corresponding to EPV = 6 inches per second, the next contour for EPV = 3 inches per second was located about 80 miles away.

The strong motion data available to McGuire were inadequate beyond a distance of about 100 miles. To estimate the attenuation of EPV beyond this distance, it was assumed that EPV at large distances from an earthquake is related to the modified Mercalli intensity (MMI). It was further assumed that the logarithm of EPV would be linearly proportional to MMI. Data from large earthquakes in California suggested that MMI decreased roughly linearly with distance, which would translate into EPV continuing to halve at equal increments of distance. Thus, the contours subsequent to those located in the previous paragraph were also spaced at about 80 miles.

For the mid-west and east, it was necessary to rely entirely on information about the attenuation of MMI (Bollinger, 1976). It appears that MMI decays logarithmically with the distance and that for the first 100 miles from a large earthquake the attenuation in these regions is roughly the same as in the west. This would imply that the distance required for EPV to halve increases with distance. Thus, starting from the contour for EPV = 6 inches per second centered on southeastern Missouri, the contour for EPV = 3 inches per second would be about 80 miles away and the contour for EPV = 1.5 inches per second would be 160 miles beyond that for 3 inches per second.

In all cases, it was stipulated that a contour for EPV should never fall inside the corresponding contour for EPA. For example, the location of the contour for EPV = 3 inches per second in south-central Illinois was determined by the contour for EPA = 0.1g rather than by distance from the contour for EPV = 6 inches per second.

After these various rules were applied to produce a set of contours for EPV, considerable smoothing was done and contours were joined where they fell close together. These steps were taken in the light of the rather meager current knowledge about EPV.

It would be highly desirable to have maps of EPV prepared using methods similar to those which have been used for peak acceleration. This was done for the northern half of California and gave results which are consistent with the contours on Figure C1-5. The maps in Figures C1-5 and C1-6 are deemed consistent with the current state-of-the-art.

G. RISK ASSOCIATED WITH EPA AND EPV

The probability that the recommended EPA and EPV at a given location will not be exceeded during a 50-year period is estimated to be about 90 percent. Given the present state of knowledge, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed by the committee after consultation with seismologists, the risk may not be just the same at all locations. It is believed that this probability of not being exceeded is in the range of 80 to 90 percent. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience, and does not imply that all buildings are thought to have a useful life of 50 years.

It must be emphasized that the 90 percent probability of not being exceeded was not established initially as a criterion for selecting the EPA and EPV. A suitable level of EPA for the more seismic regions of California was selected on the basis of various considerations, some of which have already been mentioned in Section E. Contours based on this level appeared to agree reasonably well with the level of acceleration determined by Algermissen and Perkins at the California border (California was not included in their earlier working maps), so their map was used as a guide for the rest of the country.

A probability of not being exceeded can be translated into other quantities such as mean recurrence interval and average annual risk. A 90 percent probability of not being exceeded in a 50 year interval is equivalent to a mean recurrence interval of 475 years or an average annual risk of 0.002 events per year. These other quantities have physical meaning only if averaged over very long periods of time--tens of thousands of years. In particular, a mean recurrence interval (also referred to as return period) of 475 years does not mean that the earthquake will occur once, twice, or even at all in 475 years. With present knowledge, there is no practical alternative to assuming that a large earthquake is equally likely to occur at any time, and quantities such as return period only indicate the likelihood that such an event will occur.

Figure C1-7, which is based upon information supplied by Algermissen and Perkins from their study, indicates the probabilities of not being exceeded if other levels of EPA were to be selected. For example, consider a location on the contour for EPA = 0.2g in Figure C1-3. At this location, there is about a 60 percent probability that an EPA of 0.1g will not be exceeded during a 50-year interval. Similarly, there is 98 percent probability that the EPA will not exceed 0.35g. The dashed portions of the curves indicate possible extrapolations to larger and smaller annual risks. What this upper limit might be in any seismic area and especially in the less seismic areas is a matter of great debate; some experts feel that the upper limit is the same as for highly seismic areas, although of course the probability of such an extreme EPA occurring is very, very small.

The probability that the ordinates of the design elastic response spectrum will not be exceeded, at any period, is approximately the same as the probability that the EPA and the EPV will not be exceeded. This is true because the uncertainty in the EPA and EPV that will occur in a future earthquake is much greater than the uncertainty in spectral ordinates, given the EPA and EPV. Thus the probability that the ordinates of the design elastic response spectrum will not be exceeded during a 50-year interval is also roughly 90 percent, at least in the general range of 80 to 95 percent.

H. DESIGN ELASTIC RESPONSE SPECTRA

At the present time there is a high degree of agreement that the characteristics of ground shaking and the corresponding spectra are influenced by:

1. The characteristics of the soil deposits underlying the proposed site.

2. The magnitude of the earthquake producing the design ground motions.

3. The source mechanism of the earthquake producing the ground motions.

4. The distance of the earthquake source from the proposed site and the nature of the travel path geology.

While it is conceptually desirable to include specific consideration of all four of the factors listed above it is not possible to do so at the present time due to the lack of adequate data. Sufficient information is available to characterize in a general way the effects of specific soil conditions on effective peak acceleration and spectral shapes. The effects of the other factors are so little understood at this time that they are often not considered in spectral studies. However, detailed spectral studies have shown that large portions of the response spectra can be closely represented using a scaling proportional to the EPA and EPV values (Blume et al, 1973, Newmark et al, 1973, Mohraz, 1976). The two maps can be easily used to represent the anticipated change in the shape of response spectra with the increase in distance from the seismic source zone by a direct adaptation of the response spectra for motions close to the seismic source zone.

The present recommendations therefore only consider the effects of site conditions and the distance from the seismic source zone. At such times as the potential effects of other significant parameters can be delineated and quantified, the current recommendations can be modified to reflect these effects.

Thus, the starting points in the development of the ground motion spectra are the seismic design regionalization maps that express by contours the EPA and the EPV which would be developed on firm ground.

Site Conditions

The fact that the effects of local soil conditions on ground motion characteristics should be considered in building design has long been recognized in many countries of the world. Most countries considering these effects have developed different design criteria for several different soil conditions. Typically these criteria use up to four different soil conditions. In the early part of this study consideration was given to the use of four different conditions of local site geology.

On the basis of the available body of data, the four conditions were selected as follows:

1. Rock - of any characteristic whether it be shalelike or crystalline in nature. As a general rule, such material is characterized by a shear wave velocity greater than about 2500 fps.

2. Stiff soil conditions or firm ground - including any site where soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

3. Deep cohesionless or stiff clay soil conditions - including sites where the soil depth exceeds about 2500 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

4. Soft-to-medium stiff clays or sands - characterized primarily by several tens of feet of soft-to-medium stiff clay with or without intervening layers of sand or other cohesionless soils.

Effective Peak Accelerations for Different Site Conditions

Based on the use of the four different site conditions outlined above, the values of EPA for rock conditions were first modified to determine corresponding values of effective peak ground acceleration for the three other site conditions. This modification was based on a statistical study of the peak accelerations developed at locations with different site conditions and the exercise of judgment in extrapolation beyond the data base.

After evaluating these effects and rounding out the results obtained, the values of EPA were modified as follows. For the first three soil types: rock, shallow stiff soils and deep cohesionless or stiff clay soils, there is no reduction. For the fourth soil type, soft to medium clays, a reduction factor of 0.8 is used for all seismicity index areas. It should be pointed out that the statistical data show that the reduction effect is not constant for all ground motion levels and the value of the reduction factor is generally smaller than is recommended here.

Spectral Shapes

Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of the spectral shapes developed on such soils close to the seismic source zone in past earthquakes (Seed et al, 1976; Hayashi et al, 1971). The mean spectral shapes determined directly from the study by Seed et al, based on 104 records mostly from earthquakes in the western part of the United States, are shown in Figure C1-8. These spectral shapes were also compared with the studies of spectral shapes conducted by Newmark et al (1973), Blume et al (1973), and Mohraz (1976), and studies for use in model building regulations. It was considered appropriate to simplify the form of the curves to a family of three by combining the spectra for rock and stiff soil conditions leading to the normalized spectral curves shown in Figure C1-9. The curves in this figure thus apply to the following three soil conditions.

Soil Profile Type S_1: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2500 feet per second); or stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiffer clays.

Soil Profile Type S_2: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

Soil Profile Type S_3: Soft-to-medium stiff clays and sands, characterized by 30 feet or more of soft- to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

Recommended ground motion spectra for 5 percent damping for the different map areas are thus obtained by multiplying the normalized spectra values shown in Figure C1-9 by the values of effective peak ground acceleration and the correction factor of 0.8 if Soil Profile Type S_3 exists. The resulting ground motion spectra for Map Area 7 are shown in Figure C1-10. The spectra from Figure C1-10 are shown on Figure C1-11 plotted in Tripartite form. It can be readily seen on Figure C1-11 that for all soil conditions the response spectra in the period range of about one second are horizontal or equivalent to a constant spectral velocity.

The spectral velocity values are proportional to the values of A_v given on the map for Effective Peak Velocity. For close-by motions represented by the innermost contours on the maps, spectra such as are shown on Figure C1-10 and C1-11 are applicable. Where the two sets of contour values differ, the portion of the response spectrum controlled by the velocity should be increased in proportion to the EPV value and the remainder of the response spectra extended to maintain the same overall spectral form. An example of this is shown on Figure C1-12 where the response spectra for Las Vegas and a site in South Carolina are compared. The higher response at longer periods, which is believed to be representative of motion from distant earthquakes, can be readily seen.

On the basis of the studies of spectral shapes conducted by Blume et al (1973) and Newmark et al (1973), spectra for 2 percent damping may be obtained by multiplying the ordinates of Figures C1-9 and C1-10 by a factor of 1.25.

Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67.

I. LATERAL DESIGN FORCE COEFFICIENTS

The equivalent lateral force method of design requires that a horizontal force be accommodated in the structural design. The magnitude of this force is a function of several parameters including the map area, the seismicity index, the type of site soil profile, the fundamental period of the building, and the type of building construction.

For use in a design provision or code it is distinctly advantageous to express the lateral design force coefficient in as simple a manner as possible. The recommended procedure for determining the lateral design force coefficient C_s is given in Sec. 4.2 as follows:

$$C_s = \frac{1.2 A_v S}{R T^{2/3}} \qquad \text{(C1-1)}$$

The value of C_s need not exceed $2.5A_a/R$ for Type S_1, S_2 or S_3 soils. For Type S_3 soils when A_a is equal to or greater than 0.3 the value of C_s need not exceed $2A_a/R$. The soil profile coefficient S is given in Table 3-A as:

SOIL PROFILE COEFFICIENT

	Soil Profile Type		
	S_1	S_2	S_3
Factor S	1.0	1.2	1.5

The procedure by which these curves were derived for the response spectra curves is as follows. As buildings become larger and more complex there arise, in addition to the increase in modes of vibration, many modes by which severe damage can be initiated. There is also a greater likelihood that high ductility requirements may be concentrated in a few stories of the building. These factors, when combined with the importance of larger buildings to the community, suggest that the larger and longer period structures should be given a more conservative criteria or weighting factor. It was judged that this weighting factor should make the lateral force coefficient approximately 50 percent greater at a period of 2 seconds for the stiff soil condition than would be obtained by direct use of the response spectrum. This increase should gradually reduce as the building period shortens.

The use of a simple soil factor in Formula C1-1 produces a direct approximation of the effect of local site conditions on the design requirements. This direct method eliminates the need for the estimation of a predominant site period and the computation of a soil factor based on the site period and the fundamental period of the building.

These suggested modifications could be modelled by Formula C1-1 given above including the soil profile factor S. The value of S for soil profile Type S_3, which represents a 50 percent increase over the value for stiff soil spectra, equals the maximum value of the S value in the present code. Lateral force design curves for the three soil types are shown on Figure C1-13. These have been computed directly from the above relationships with the values of A_a, A_v, and R taken as 1.0. A comparison between the lateral design force coefficients and the free field ground motion spectra is shown on Figure C1-14.

In the application of these recommendations the values of A_a and A_v may not be equal, so that the lateral force coefficient curves will be different from those discussed above. To illustrate the varying effects obtained from the use of the lateral force equation, the respective curves of C_sR for shallow stiff soil sites for several cities are shown on Figure C1-15.

J. COUNTY-BY-COUNTY MAPS

It is generally recognized that the exposure to seismic hazard reduces as the distance from an active seismic region increases. It was in recognition of this simple premise that among the first recommendations in the ATC-3 project was abandonment of the broad uniform seismic zoning presently being used. The first recommendation suggested that seismic zoning should be on the basis of the contours shown on Figures C1-3, C1-4, C1-5, and C1-6, with interpolation being used to obtain values between the contour levels. It soon became apparent that interpolation by the user would produce some difficulties in coastal areas and along the international borders where interpolation would require extension of the contours beyond the national boundaries. This difficulty combined with the problem of defining a simple interpolation procedure with no ambiguity led to an alternate method of producing zoning maps; the use of Map Areas with specified values of A_a or A_v with boundaries along those of political jurisdiction. The simplest form of subdivision for the contiguous states was by the use of county boundaries.

The county by county seismic design regionalization maps are presented in Chapter 1 of the provisions as Figures 1-1 and 1-2 and are used to determine the A_a and A_v coefficient values respectively. The county by county maps were prepared by assuming that each county should be represented by the highest contour in that county. In developing the county-by-county map, intermediate contours were drawn for coefficient values of 0.3 and 0.15 which are listed in Table 1-B but are not shown on Figures C1-3 and C1-5. It can be seen that the procedure of assigning the same value throughout a county produces discontinuities in some areas of the map. In such areas it is strongly recommended that local jurisdictions consider microzonation of their counties which extend across several different contour values.

A Seismicity Index is included in these provisions. The Seismicity Index is intended to reflect the ability of different types of construction to withstand the effects of earthquake motion. This Index is related to the toughness or energy-dissipation characteristics of the construction type used and provides a means in Table 1-A of determining which construction types are permitted in each of the map areas. It is recognized that damaging seismic motion can be better correlated by using velocity rather than acceleration and therefore the Seismicity Index is determined from the map values for EPV in accordance with Table 1-B of the design provisions. It should be noted that the Seismicity Index values are different in Map Areas 1 and 2 although the A_v values are the same. A minimum value of A_a and A_v of 0.05 was used throughout and designated as Map Area 1. Where the seismic risk procedure produces a value of 0.05 the Map Area value is changed to 2 and the Seismicity Index becomes 2.

The values of the coefficients A_a or A_v and the Seismicity Indices associated with map areas are as follows:

MAP AREA	VALUE OF COEFFICIENT A_a	A_v	SEISMICITY INDEX
7	0.40	0.4	4
6	0.30	0.3	4
5	0.20	0.2	4
4	0.15	0.15	3
3	0.10	0.10	2
2	0.05	0.05	2
1	0.05	0.05	1

K. COST IMPLICATIONS

The effect of the proposed design provisions upon the intial cost of buildings has as yet not been evaluated by costing actual designs. However, experience with previous codes provides considerable guidance. This question is enormously complex and it is possible to arrive at many different answers depending upon:

- The role in society of the person answering.
- Whether the building already exists or has yet to be designed.
- Whether or not the building is required to remain functional after a major earthquake.
- Whether or not some seismic design requirements already apply to the building.

Building costs

First consider the case of new construction which is not subject to the requirement of remaining functional following an earthquake. Some quantitative data have been developed for this case, derived in part from the upgrading of previously applicable seismic design requirements (ATC-2 Report, 1972) and in part from buildings which heretofore have not been subject to any such requirements (SEAOC, 1970; Whitman et al, 1975; Larrabee and Whitman, 1976). The major factors influencing the increased cost of complying with the tentative provisions are:

1. The complexity of the shape and structural framing system for the building. It is much easier to provide seismic resistance in a building with a simple shape and framing plan.

2. The cost of the structural system (plus other items subject to special seismic design requirements) in relation to the total cost of the building. In many buildings, the cost of providing the structural system may be only 25 percent of the total cost of the project.

3. The stage in design at which the provision of seismic resistance is first considered. The increased cost can be inflated greatly if no attention is given to seismic resistance until after the configuration of the building, the structural framing plan, and the materials of construction have already been decided.

Obviously, the increased cost can vary enormously from project to project.

In the best case--a simple building with short spans where earthquake requirements are introduced at a very early stage of project planning--the increased cost of the structural system should be in the range of 1 to 8 percent of the cost with no seismic design requirements. The highest increase would apply to a building subject to the most severe of the tentative provisions. In terms of total project cost, the increase would be from about one-half percent to perhaps 5 percent.

In the worst case--a complex, irregular building with long spans, where earthquake requirements are considered only after the major features of the design are frozen--the increases can be considerably greater; perhaps as large as 25 percent of the structural cost as compared to the same building with no seismic resistance.

A requirement that a building remain functional after an earthquake will generally mean still greater structural costs and also costs to provide earthquake resistance for architectural, electrical, and mechanical systems and equipment. Because there has been only limited experience with the impacts of such requirements, it is not possible to provide figures on the probable amounts of such increases.

The costs of upgrading the seismic resistance of existing buildings can also be much greater. Experience in California suggest increases on the order of $5 per square foot (1976 prices) for measures which improved resistance but fell short of full compliance with the resistance required in new construction, to on the order of $10 to $15 or more per square foot where full compliance with existing (Circa 1970) codes was achieved. Still larger costs have been incurred in some projects where there were special requirements to preserve the exterior appearance of a building, etc.

Other Costs

The costs quoted above are of greatest interest to the owners of a proposed building. There are other potential cost implications of the tentative provisions, each of which reflects the viewpoint of different groups in society.

For an engineering firm, any change in required design requirements means some overhead cost--in learning to interpret the requirements, altering standard procedures and computer programs, and updating standard design details. Some qualitative information regarding these costs appears in the ATC-2 Report. While such costs can be significant in the short run, in the long run they become modest.

Any change in design requirements also has potentially significant costs to suppliers of building materials and of proprietary building systems. In the short run, changes may adversely affect the competitive advantage of an organization or industry. In the long run, however, American industry has always shown remarkable adaptability to new building regulatory requirements.

Adoption of new design requirements may mean additional costs to city, state, and federal agencies charged with administration and enforcement of the requirements. Such agencies are in a position similar to that of an engineering firm.

Perspective on Cost

On the average, the increased cost of buildings conforming with these tentative provisions should be less than one percent of the total cost of the building. This should be equally true in areas of low to moderate seismicity which have not previously required design against earthquakes, -and areas of high seismicity that have had such requirements. For buildings which must remain functional after an earthquake, the cost increases may be greater.

The transient costs associated with a change in design requirements should diminish or disappear with time.

L. IMPLIED RISK

This commentary Section discusses methods for evaluating implied risk and presents one estimate of the risk implied by the tentative seismic design provisions. The word "risk" is used here in a general sense to indicate losses that may occur in the future, at uncertain time and in uncertain amounts as a result of earthquake ground shaking.

It is not possible by means of a building code to provide a guarantee that buildings will not fail, in some way that will endanger people, as a result of an earthquake. While a code cannot ensure the absolute safety of buildings, it may be desirable that it should not do so as the resources to construct buildings are limited. Society must decide how it will allocate the available resources among the various ways in which it desires to protect life safety. One way or another, the anticipated benefits of various life protecting programs must be weighed against the cost of implementing such programs.

One reason a code cannot ensure absolute safety is the present (and probably future) inability to describe on firm scientific ground the strongest earthquake ground shaking that might possibly occur at any specified location. As long as it is not possible to describe the largest possible ground shaking, it is impossible to design for zero risk. Hence, a decision to design a building for a specified capacity has associated with it an implicit risk. This implied risk may be quite small (e.g. 1 chance in 10,000 that a building will fail during an earthquake), but it is greater than zero.

None of the methods or estimates presented in this Section are precise. Indeed, they are quite crude and quite uncertain. However, the methods and estimates serve two very valuable purposes. First, they show the factors and considerations that influence overall risk. Second, they give a general indication of the level of safety provided by the tentative seismic design provisions in comparison with other risks faced by society.

Expressing Losses

In general, losses may be in the form of damage and repair costs, injuries and fatalities, and the indirect adverse effects upon a community, region, or country. Because the emphasis of the proposed seismic design provisions is upon life safety, this Section is specifically concerned with losses directly related to life safety. In many ways it might be more appropriate to use injuries and fatalities, i.e. "major casualties", as a measure of the risk to life safety. Because many people find it difficult to talk in terms of predicted major casualties and it is difficult to make accurate predictions concerning major casualties, this Section will make use of an indirect measure of the risk to life safety--the risk of failure of buildings, where such failure would imply a threat to life safety. More precise definitions of failure will be discussed subsequently.

Expressing Probability

The time when the next major earthquake will affect a particular city is unknown, as is the magnitude of that earthquake. The future losses sustained in that city may result from several moderate-sized earthquakes or from a single large earthquake. Since there is little agreement as to the specific nature of the most intense ground shaking that might occur, especially in the less seismically active parts of the country, it is difficult to be specific about the largest possible losses that might occur. These considerations mean that the future losses are uncertain and some measure of probability must be used in the examination of such losses. While this might be done in several ways, there are two commonly used approaches.

One way is the use of average annual losses. Risk might be expressed as the average dollar loss per year, the average major casualties per year, the average number of building failures per year, etc. Losses expressed in this way are annual risks. However, large earthquakes are very rare events, and losses averaged for such infrequent events may not give a meaningful portrayal of the large loss that might occur for one such event.

The second way is to define a threshold of loss and to estimate the probability that the threshold will be equalled or exceeded during some earthquake. For example, we might speak of the probability that the dollar cost of damages and repairs will exceed one billion dollars during at least one earthquake during the next fifty years. The threshold might alternatively be some number of human casualties or some number of building failures.

Estimating Probability of Failure - General Procedure

The design earthquake ground motion by itself does not determine risk; the risk is also affected by the design rules and analysis procedures used in connection with the design ground motion. That is, the overall risk to a building is determined by both the seismic hazard and the probable building performance. This is expressed by the following equation giving the average number of failures, f, per year for an individual building.

$$f = P[F|a]\frac{d\gamma}{da}da \qquad \text{(C1-2)}$$

where

a = the EPA or EPV as appropriate
P[F|a] = the probability of failure if an intensity of shaking with EPA = a occurs,
γ = the annual rate at which intensities of shaking are exceeded (see Figure C1-7).

The integration is over all possible values of a. The average annual rate of failures can then be converted to the probability that failure will occur during some period of time. This is the same as the conversion between the left-hand and right-hand scales of Figure C1-7.

Estimated Performance of Buildings Designed According to The Tentative Provisions

The following paragraphs give rough estimates, based on experience and judgment, of the probability of failure occurring when a building designed in accordance with the tentative provisions is subjected to different levels of ground shaking. However rough, the estimates should suffice for general guidance as to the degree of safety implicit in the tentative provisions. The estimates are intended to apply to a building of moderate size and complexity, meeting the minimum requirements of the provisions.

If the design ground motion were to occur, structural collapse--meaning collapse of part and in extreme cases of all of a building -- should not be expected in buildings designed in accordance with the provisions. (Failures due to design or construction errors cannot be prevented by design requirements alone; detailed design reviews and mandatory construction inspection are also necessary.) If ground motions twice as strong as the design ground motions were to occur, there might be structural collapses in about 1 to 2 percent of the buildings designed in accordance with the provisions. If a ground motion is three times as strong as the design earthquake motions, this percentage might be 5 to 10 percent.

If the design ground motion were to occur, there might be life-threatening damage in 1 to 2 percent of buildings designed in accordance with the provisions. (In each building so damaged, on the average, about 1 percent of the occupants might be major casualties.) If ground motions two or three times as strong as the design ground motions were to occur, the percentage of buildings with life-threatening damage might rise to about 10 to 50 percent respectively.

These estimates are presented in graphical form in Figure C1-16, to illustrate the expected performance of buildings designed for different EPA. Possible extrapolations of the relations are suggested. The extrapolation toward low conditional probabilities of failure is difficult to estimate; in effect, one is asking; what is the probability of such major design and construction errors that it might "fail" during a very small ground motion?

Implicit Risk for a Single Building

The information contained in Figures C1-7 and C1-16 has been used as input to Formula C1-2, to compute failure probabilities for four buildings: one located on the contour in Figure C1-3 for 0.4g and designed for that EPA; one on the contour for 0.2g and designed for that EPA; and likewise for buildings located on the 0.10g and 0.05g contours. In each case, several different assumptions were made as to how the solid line in Figures C1-7 and C1-12 should be extrapolated.

It was found that, because of compensating trends, the probabilities of failure were roughly the same for each of the buildings. For buildings on the contours for 0.05g and 0.10g, the result is influenced strongly by the way in which the curves of Figures C1-7 and C1-16 are extrapolated to larger values of EPA or EPV. On the other hand, the results for a building located on the contour for 0.4g are influenced strongly by the extrapolations to smaller values of EPA or EPV.

Table C1-1 gives estimates for the probability that the two types of failure will not occur within a 50-year period. Note that these probabilities are more favorable than those for the design EPA or EPV. This simply means that a building generally will not fail just because the shaking in some earthquakes slightly exceeds the design EPA.

It must be emphasized that these estimates are very crude. All of the potential difficulties discussed in Section E apply even more strongly here.

Implicit Risk for a Group of Buildings

If there are a number of similar buildings at some location, such that all buildings experience approximatley the same shaking during any one earthquake, the probability that at least one of the buildings will fail is greater than the probability that any one particular building will fail. Calculations have also been made for this case, assuming 100 similar buildings. Results are included in Table C1-1. This case represents, in a very crude sort of way, the expected performance in any one city of new construction designed and constructed in accordance with these provisions.

When one considers a series of cities, the probability that at least one failure will occur becomes even greater. To illustrate this, assume five cities each having 100 buildings designed in accordance with these provisions. From Table C1-1 it is seen that the probability of a failure occurring is no longer insignificant.

These results emphasize that the perception of the level of safety achieved by the provisions is different for the owner of a single building, the public officials of a city, or the public officials of a state.

TABLE C1-1

PROBABILITY OF NOT HAVING ANY FAILURES
DURING A 50-YEAR PERIOD

	Type of Failure	
	Life-Threatening Damage	Structural Collapse
Single building	99%	99-99.9%
100 buildings - 1 city	90%	95%
100 buildings - 5 cities	65%	85%

M. ACCEPTABLE RISKS

There are no laws in the United States that state an "acceptable number" of fatalities per person exposed per year, or any other proposed definition of acceptable risk. Nor are there judicial decisions that give firm guidance. Legislative bodies have chosen alternatives with implied risks that have been stated quantitatively. For example, in arriving at new seismic requirements for existing buildings the Long Beach City Council opted for an alternative to which a risk of 10^{-6} fatalities per person exposed per year had been attached (the other alternatives implied smaller risks). Obviously there have been many other cases where legislative, judicial, and executive bodies have made choices which imply some level of risk. However, all such instances taken together do not constitute a firm set of precedents.

There have been attempts to determine an acceptable level of risk on fundamental grounds. As one example, Wiggins (1975) compiled data for the risk in situations (driving, flying commercial airlines, accidents in the home) where people more-or-less knowingly exposed themselves to risk. These so-called voluntary risks are of the order of 200 fatalities per million people exposed per year. Then Wiggins referred to the work of Starr (1969), who concluded that the public wants involuntary risks (such as from earthquakes) to be much smaller (say 100 to 10,000 times smaller) than voluntary risks. Thus the acceptable risk from earthquake might be between 1 and 0.01 fatalities per million people exposed per year.

As a second example, Figures C1-17 and C1-18 summarize data for the probability of man-made and natural disasters causing greater than various numbers of fatalities. Obviously, these data reflect past practice and not necessarily levels of risk that are desirable. If the "total man-caused" and "total natural" curves are reduced by 1000 (so as to give a level of risk that would not contribute significantly to total overall risk), for a 50-year exposure period, there would be a 2.5 percent probability of one or more such events.

The analysis in Section L can be used, in a crude way, to.provide risk estimates for comparision with Figures C1-17 and C1-18. Consider buildings of moderate size housing several hundred people, such that a structural collapse would--considering that buildings are usually unoccupied or lightly occupied for much of a week--on the average cause 100 fatalities. For the case of five cities with 100 buildings in each city, the frequency of an earthquake causing about 100 fatalities was estimated to be 0.003 events per year. With 50 cities with 100 such buildings each, the rate rises to 0.03 events per year. To the extent that this calculation is valid, it might then be concluded that the recommended code provisions are not unduly conservative.

Another approach to determining an appropriate level of risk is by a cost/benefit analysis. Such analyses are difficult when lives are at stake, but can be applied to the prospective loss aspect of earthquake damage. While the recommended provisions have been written to minimize the hazard to life safety, as a by-product they will reduce damage costs--especially during moderate-sized earthquakes. In highly seismic areas where moderate earthquakes occur frequently, any increase in building costs will be offset by reduced costs of damage. In less seismic areas, however, seismic design requirements can be justified only in terms of life safety; the expected savings in damage during very infrequent earthquakes are not great enough to justify an average 1 percent increase in building costs.

Other Viewpoints

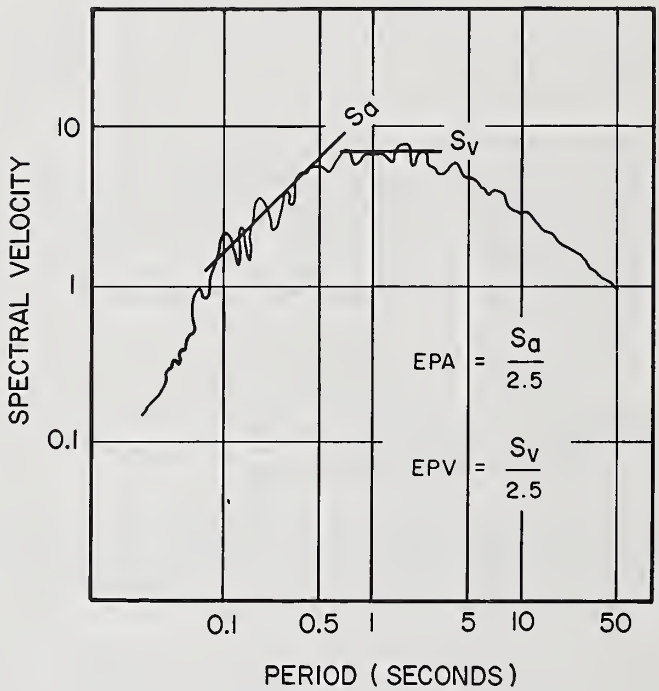
The technical approaches described in the previous paragraphs are useful in helping to decide whether or not the level of risk implicit in a proposed course of action is acceptable. However, these approaches do not by themselves make such decisions. Rather, they are made through legislative, administrative, and judicial processes.

In proposing and enacting legislation, administrative and legislative bodies have increasingly expressed interest in results from technical cost/benefit and risk/benefit studies. However, such bodies make it clear that they do not wish to be bound by results of such studies, and it is understandable that any administrator or legislator would be very hesitant to explicitly endorse any non-zero risk of fatalities as being acceptable. Ultimately, administrators and legislators are guided by their own perspectives of the wishes of society.

Society--the mass of people--makes its decisions based on fragmented information and from many varying viewpoints. The people, individually and collectively, simply do not perceive risk in a quantitative manner which can even relatively be correlated. Society is strongly influenced by credible leaders. To the extent that such leaders are influenced by technical analyses, society is indirectly influenced by them.

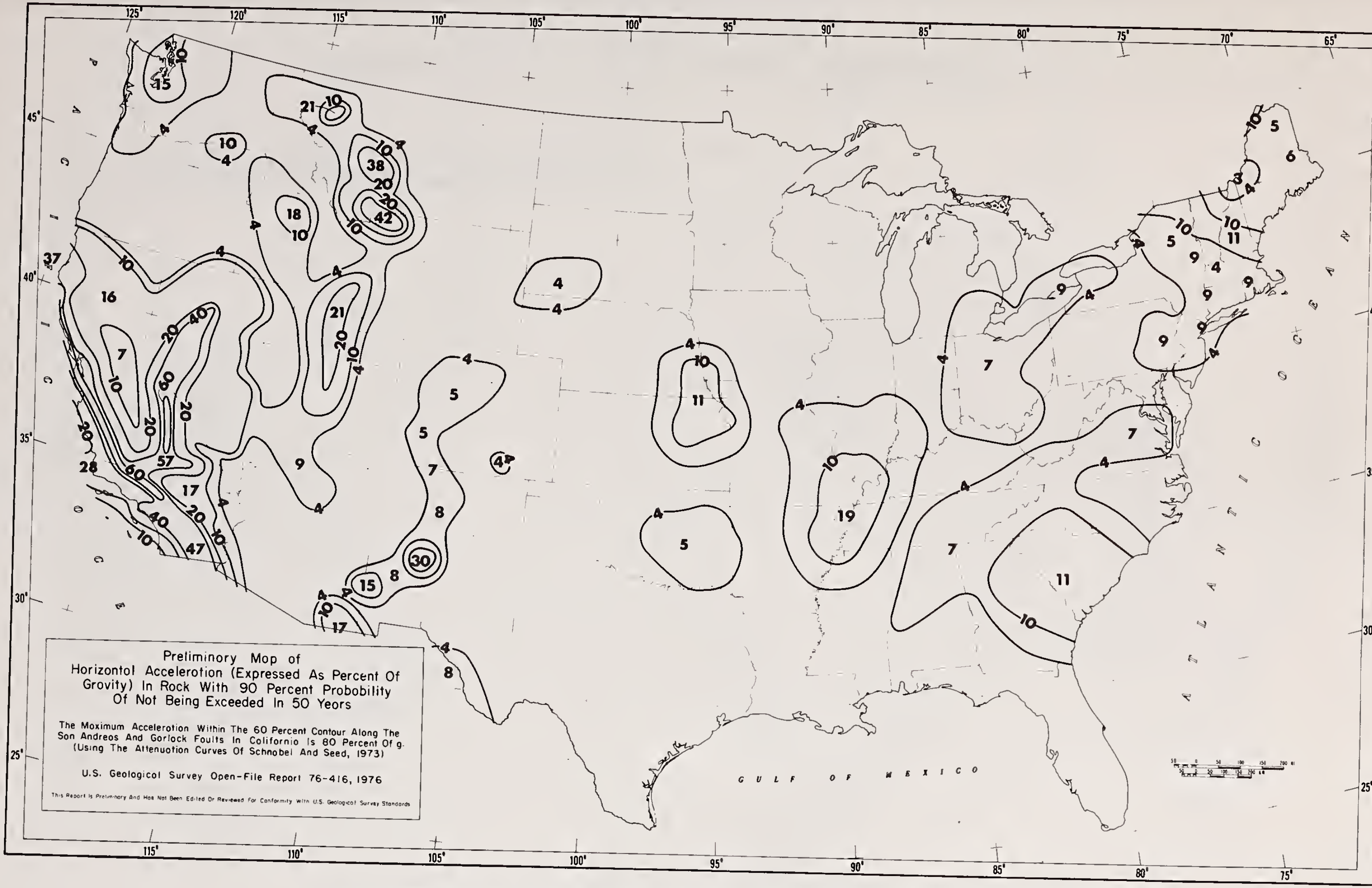
Administrative bodies have the task of interpreting legislation so as to know how to apply it, and the act of interpretation implicitly involves decisions about acceptable risk. In this role, administrative bodies evaluate their risk by relating administrative directives to the ultimate in peer practice.

Often the courts become the final judge of whether a proposed course of action for mitigating a hazard is acceptable. The body of law that has been developed in the area of flood plain regulation is a useful guide to judicial reactions to hazard mitigation. The lesson is to match severity of the regulation to the severity of the risk. The courts follow the principle of the reasonable person who strives to achieve this balance, and uses data to support findings of the appropriate balance.



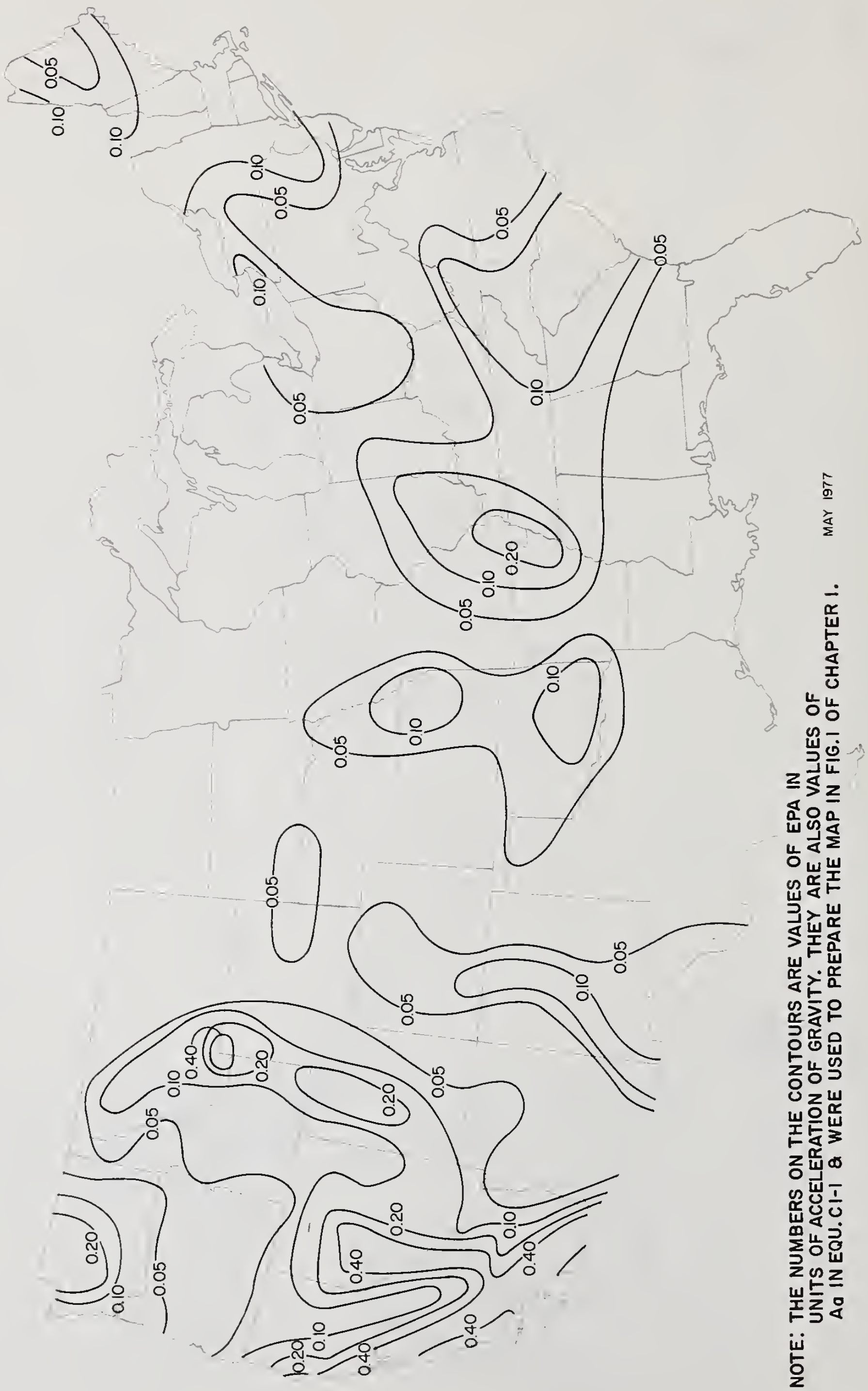
SCHEMATIC REPRESENTATION
SHOWING HOW EFFECTIVE PEAK ACCELERATION
AND EFFECTIVE PEAK VELOCITY
ARE OBTAINED FROM A RESPONSE SPECTRUM

FIGURE C1-1



SEISMIC RISK DEVELOPED BY ALGERMISSEN AND PERKINS

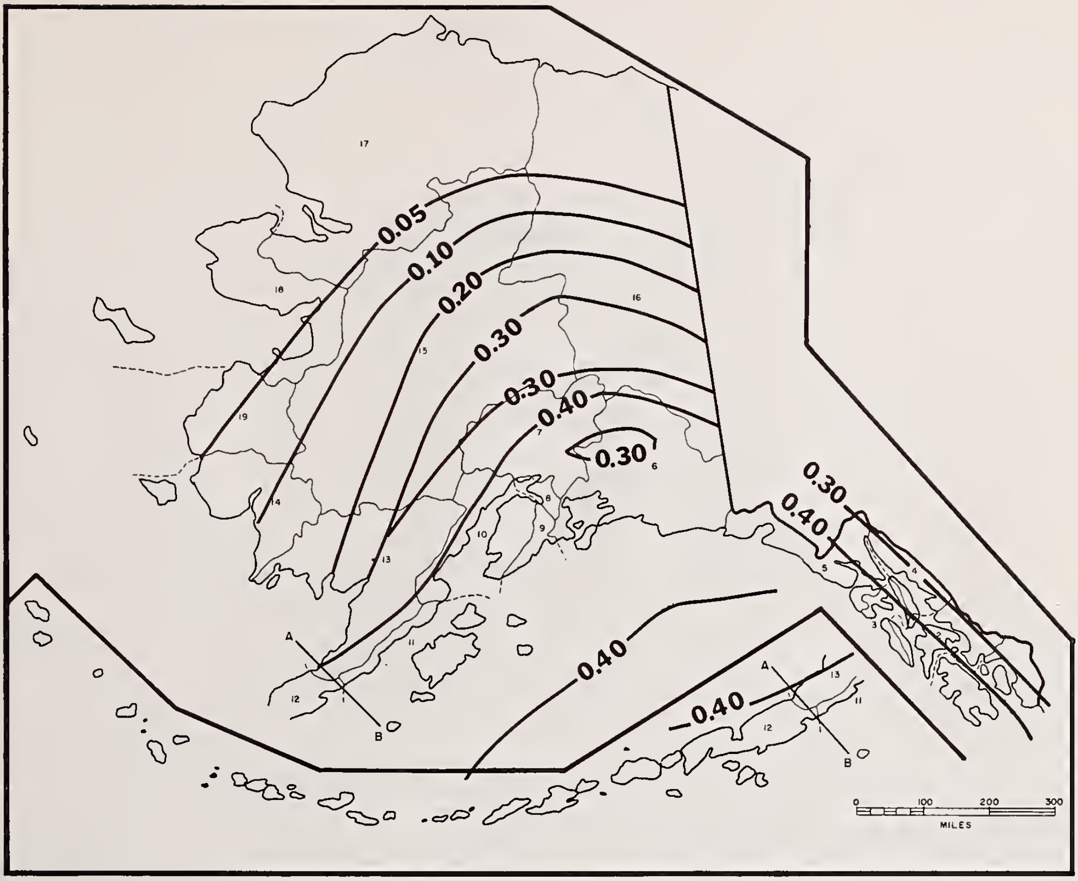
FIGURE C1-2



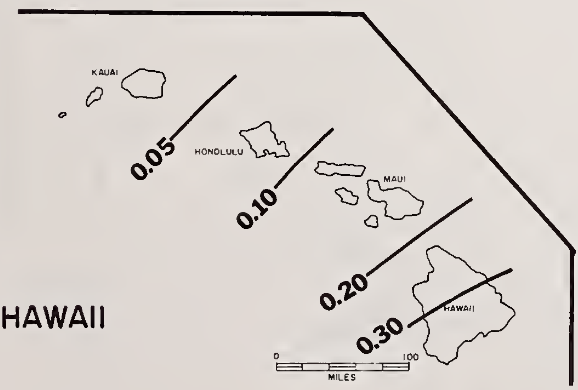
NOTE: THE NUMBERS ON THE CONTOURS ARE VALUES OF EPA IN UNITS OF ACCELERATION OF GRAVITY. THEY ARE ALSO VALUES OF Aa IN EQU. C1-1 & WERE USED TO PREPARE THE MAP IN FIG. 1 OF CHAPTER 1.

CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION

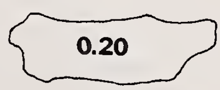
FIGURE C1-3



ALASKA



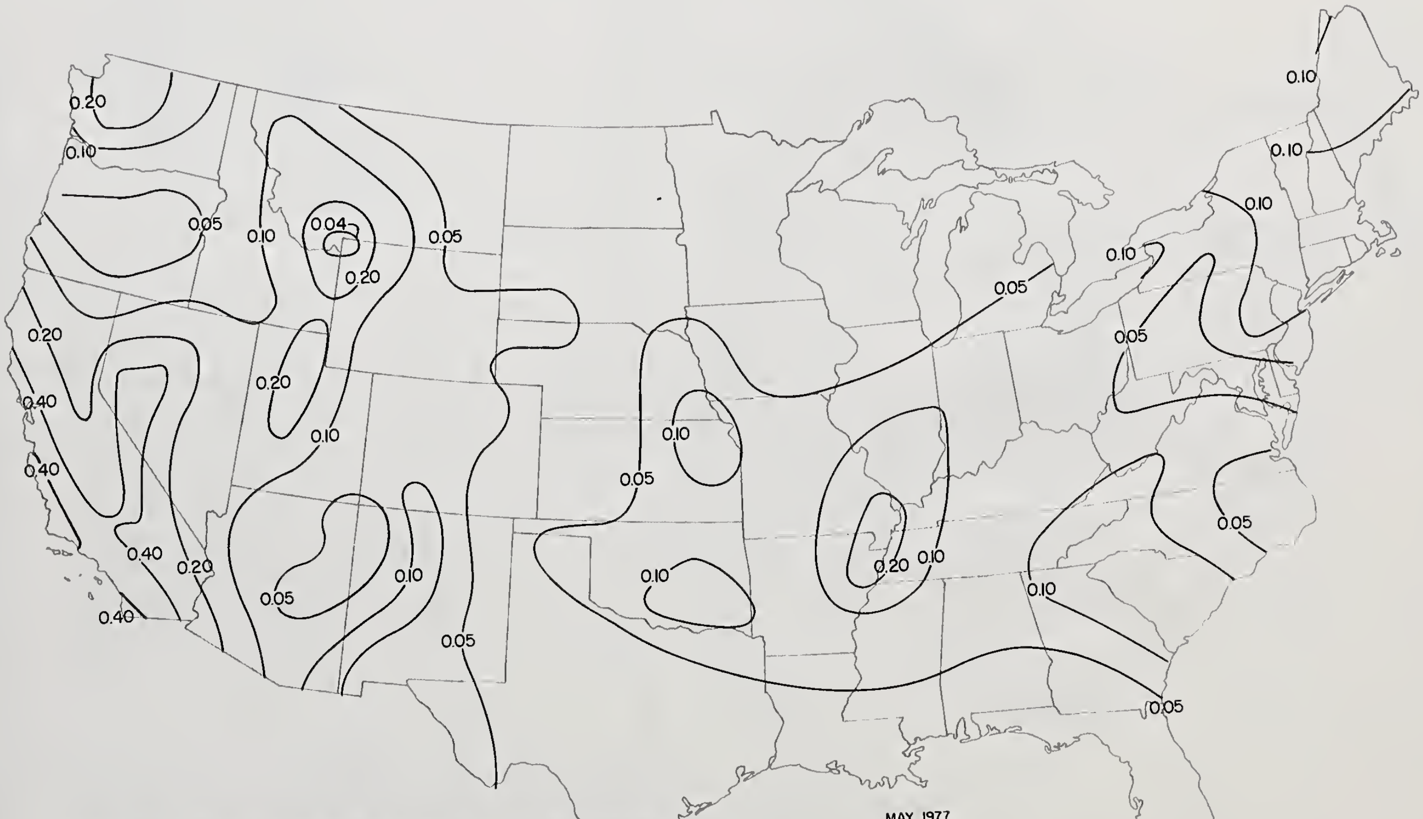
HAWAII



PUERTO RICO

CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION

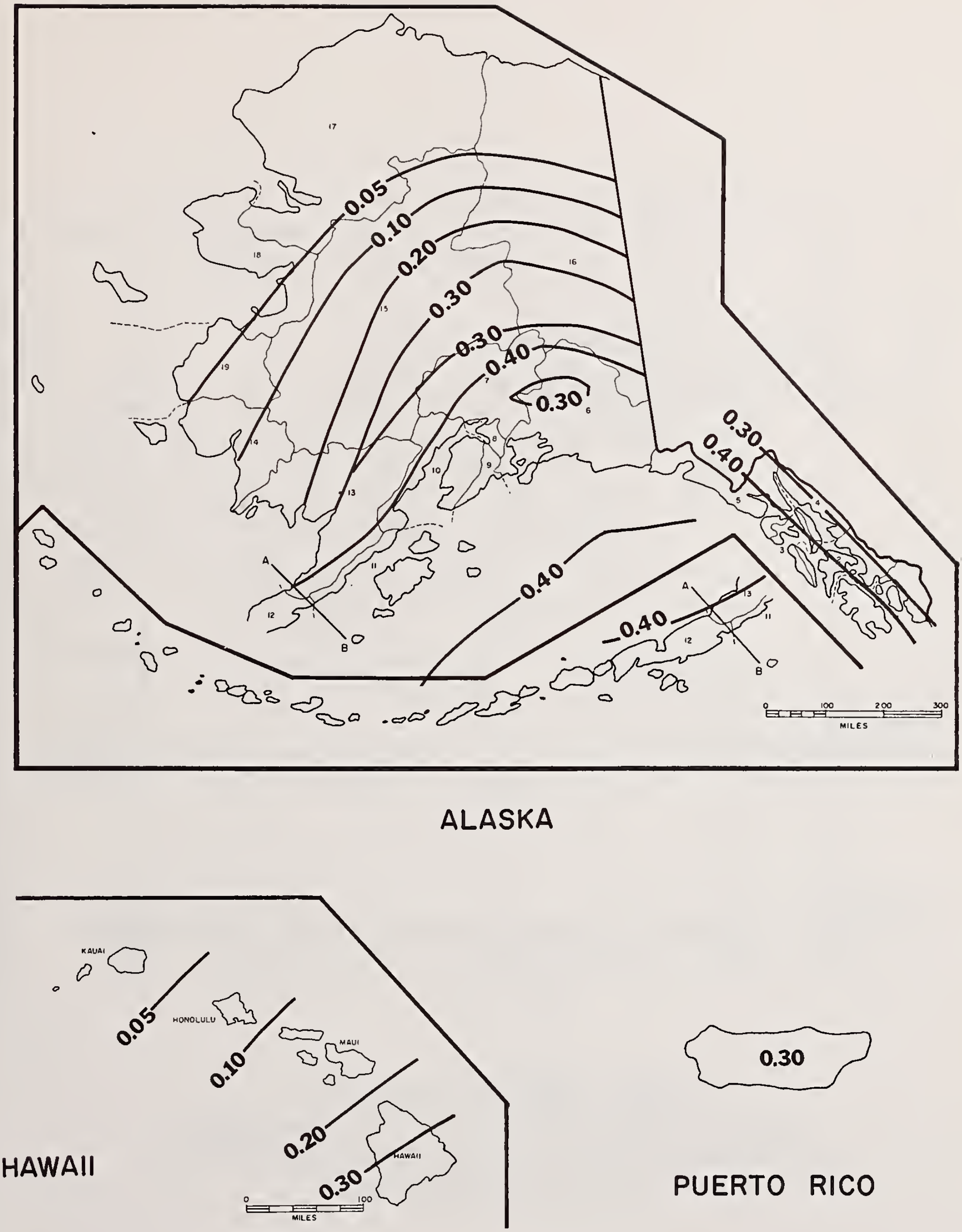
FIGURE CI-4



NOTE: CONTOURS SHOW VALUE OF A_v FOR USE IN EQUATION CI-1

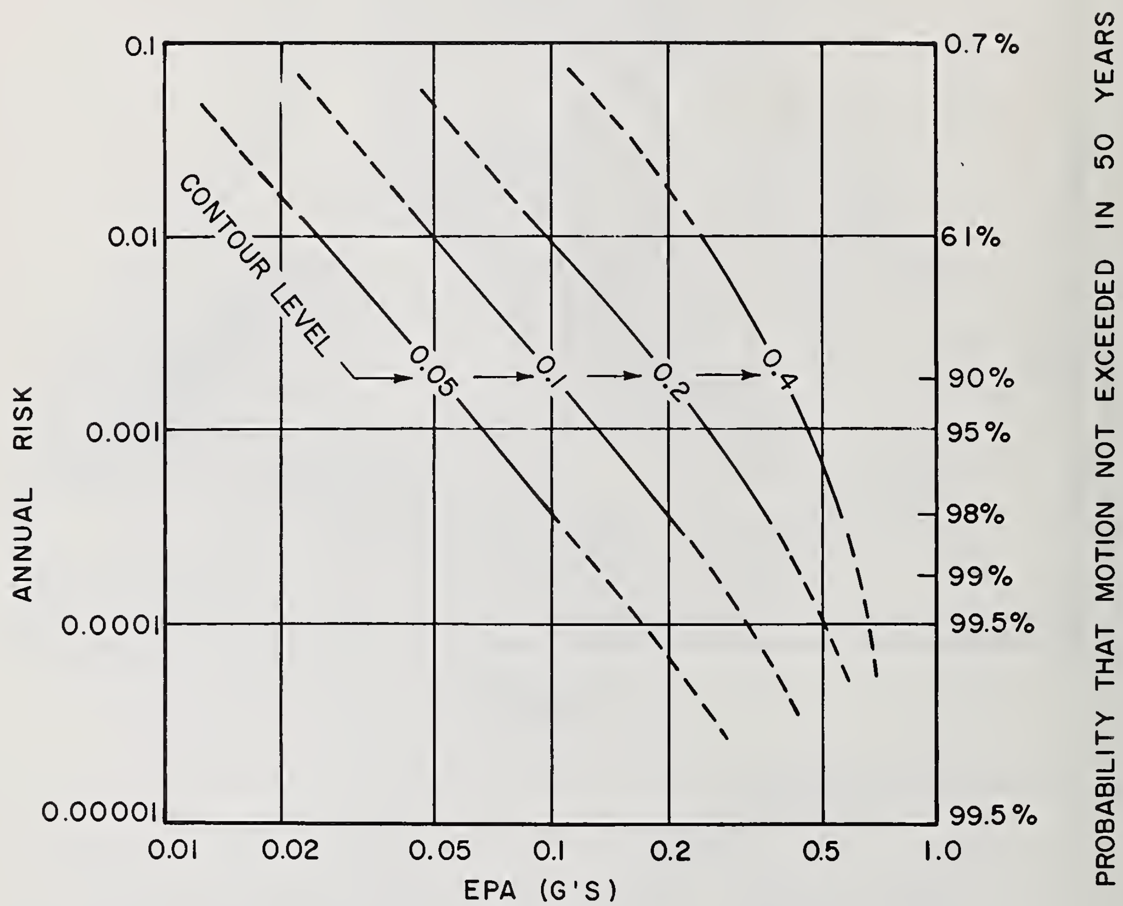
CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT

FIGURE CI-5



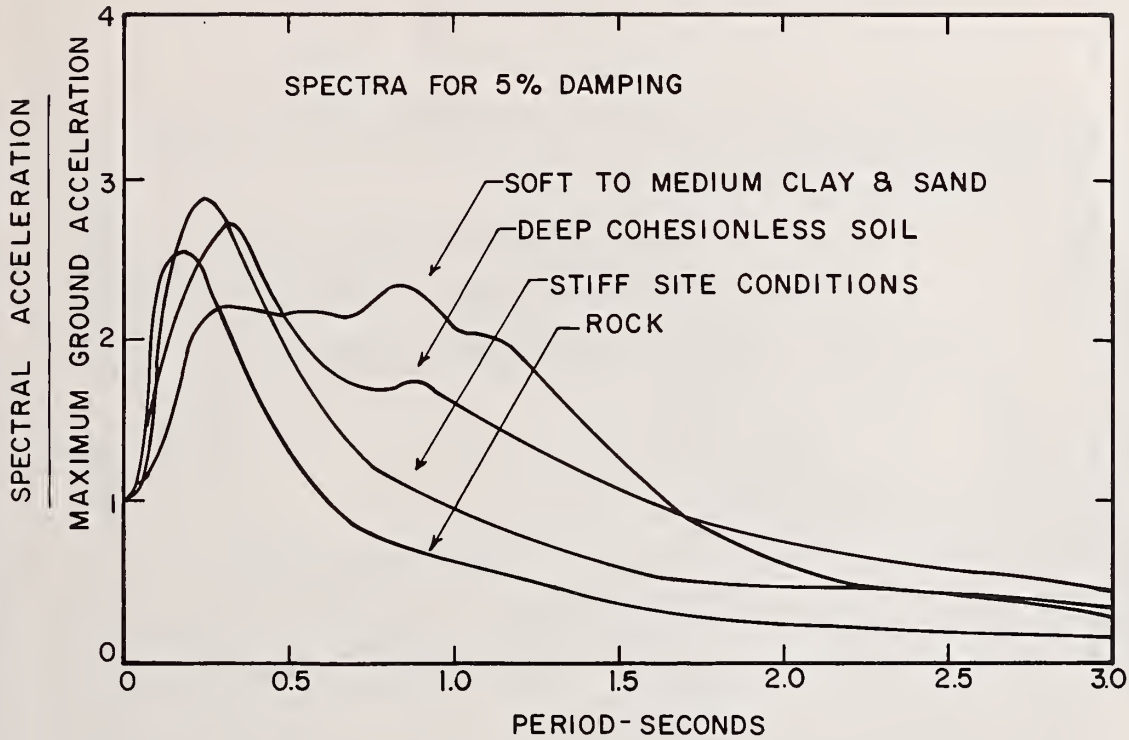
CONTOUR MAP FOR EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION COEFFICIENT

FIGURE CI-6



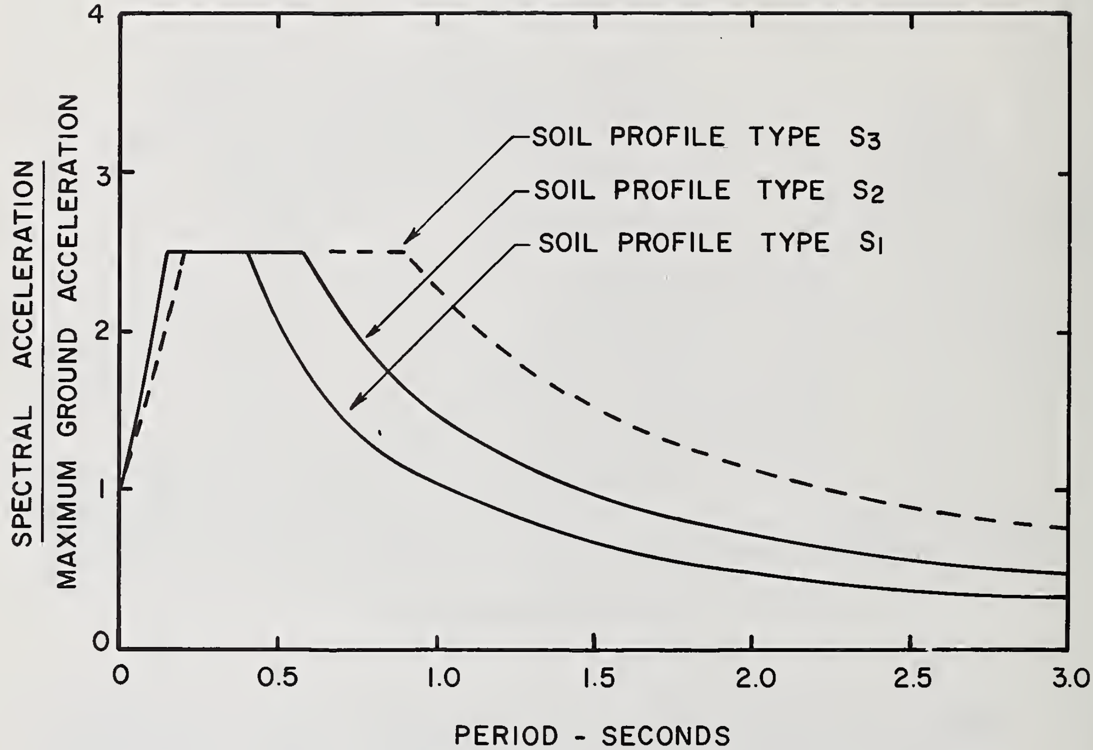
ANNUAL RISK OF EXCEEDING
VARIOUS EFFECTIVE PEAK ACCELERATIONS
FOR LOCATIONS ON THE INDICATED CONTOURS
OF EPA IN FIGURE CI-3

FIGURE CI-7



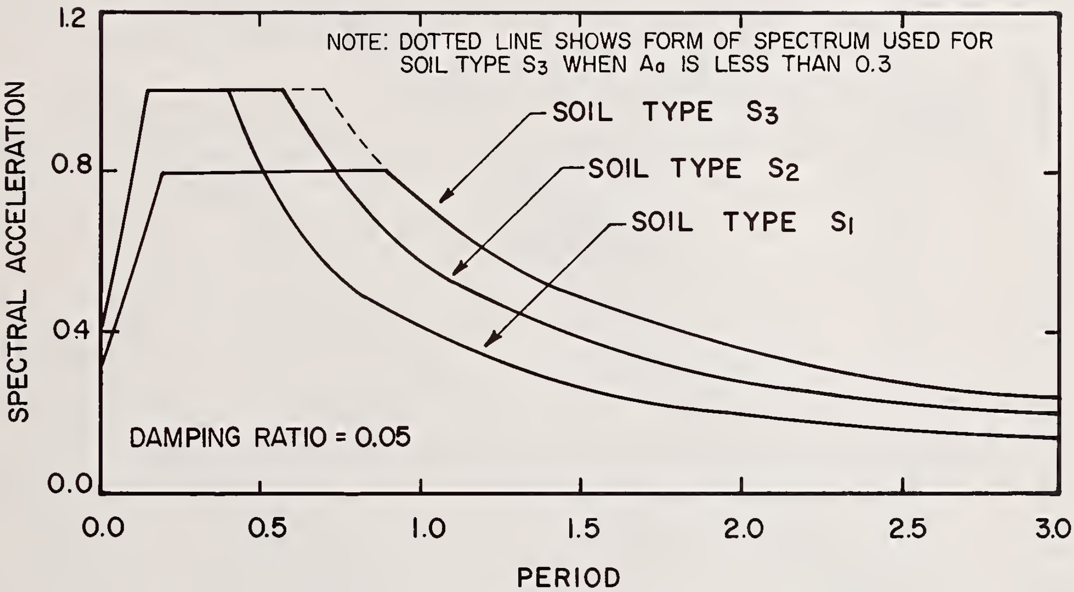
AVERAGE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS. (after SEED et al 1976)

FIGURE C1-8



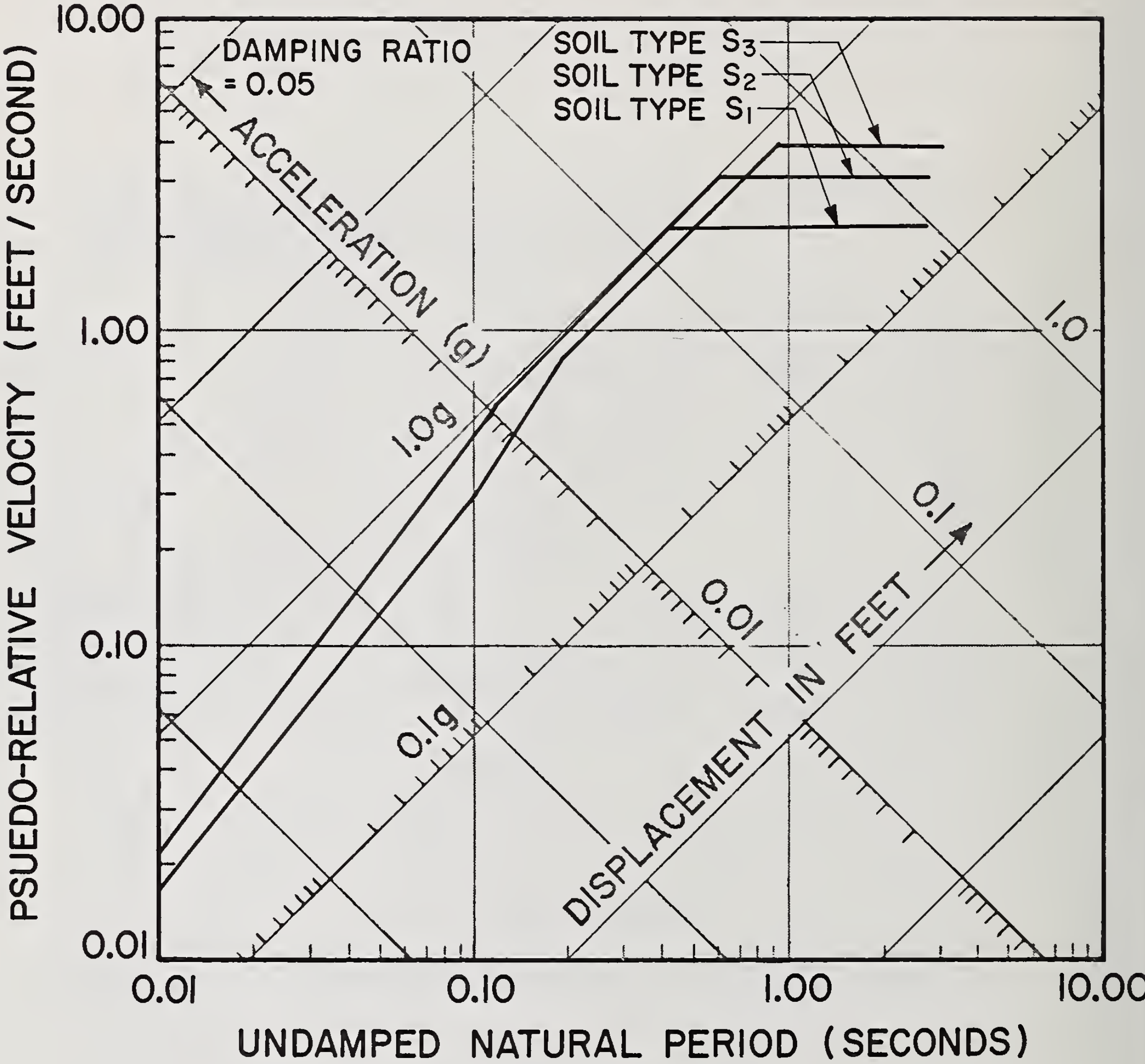
NORMALIZED RESPONSE SPECTRA
RECOMMENDED FOR USE IN BUILDING CODE

FIGURE C1-9



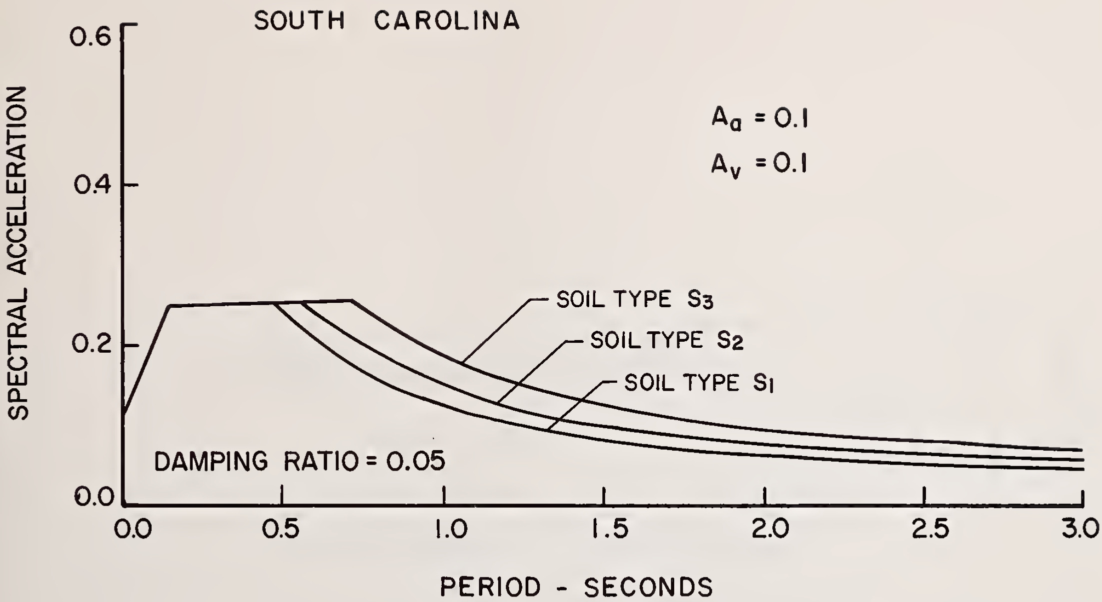
GROUND MOTION SPECTRA FOR MAP AREA 7 (A_a = 0.4)

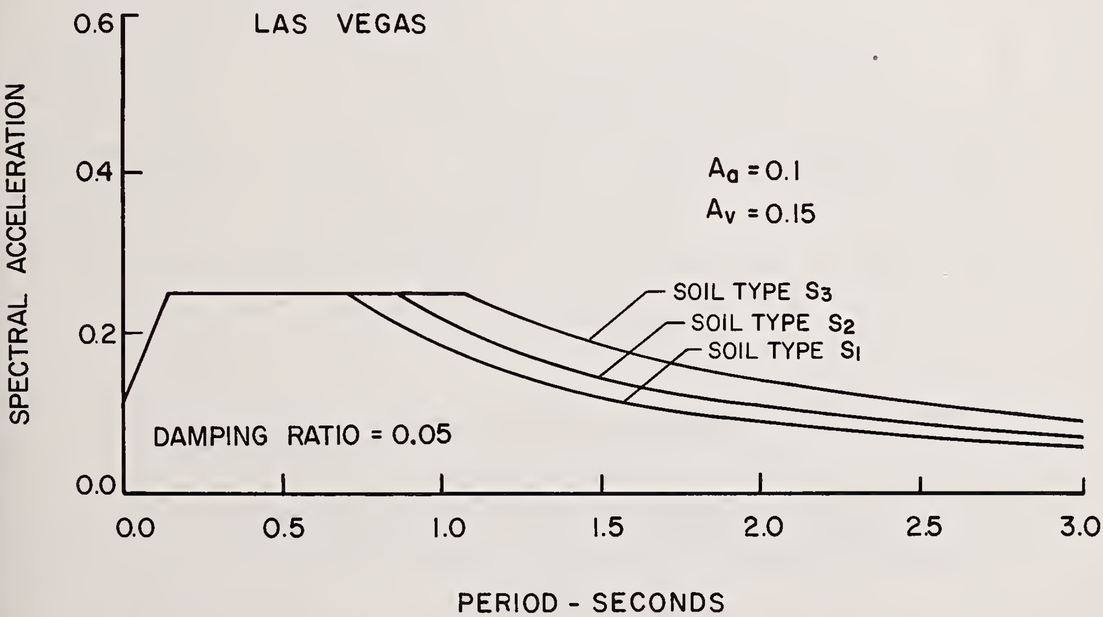
FIGURE C1-10



GROUND MOTION SPECTRA FOR MAP AREA 7 (A_a = 0.4)

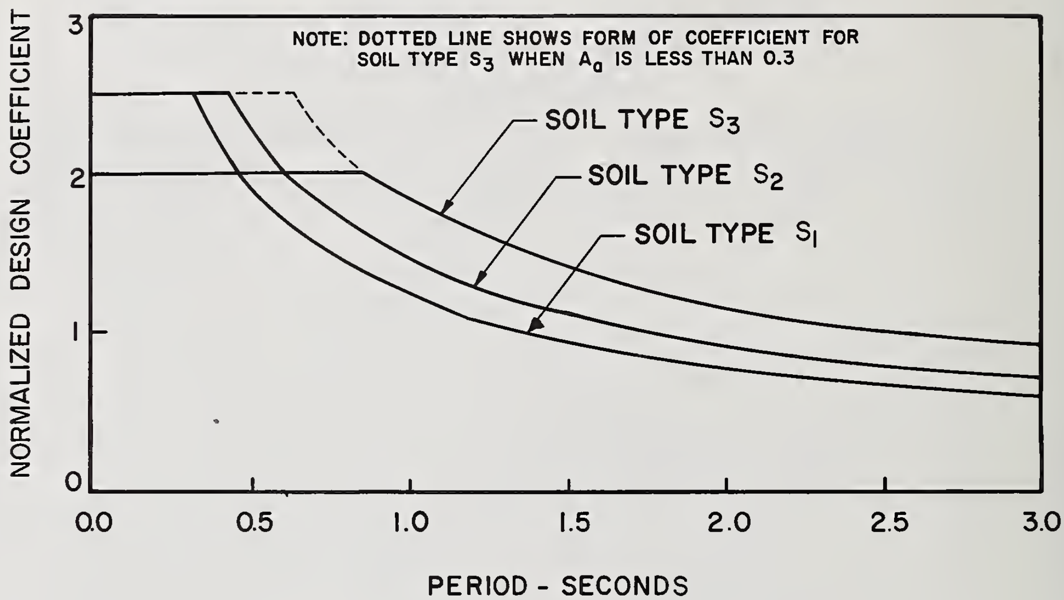
FIGURE C1-11





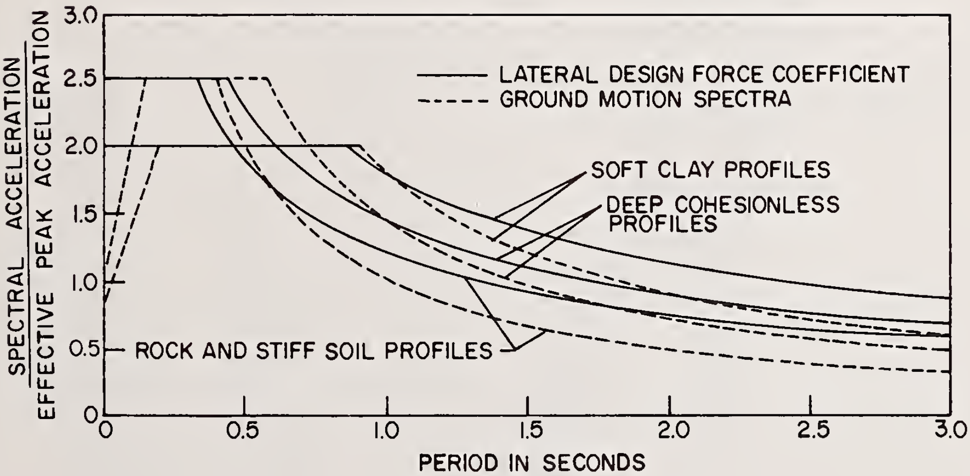
EXAMPLES SHOWING VARIATION OF GROUND MOTION SPECTRA IN DIFFERENT TECTONIC REGIONS

FIGURE C1-12



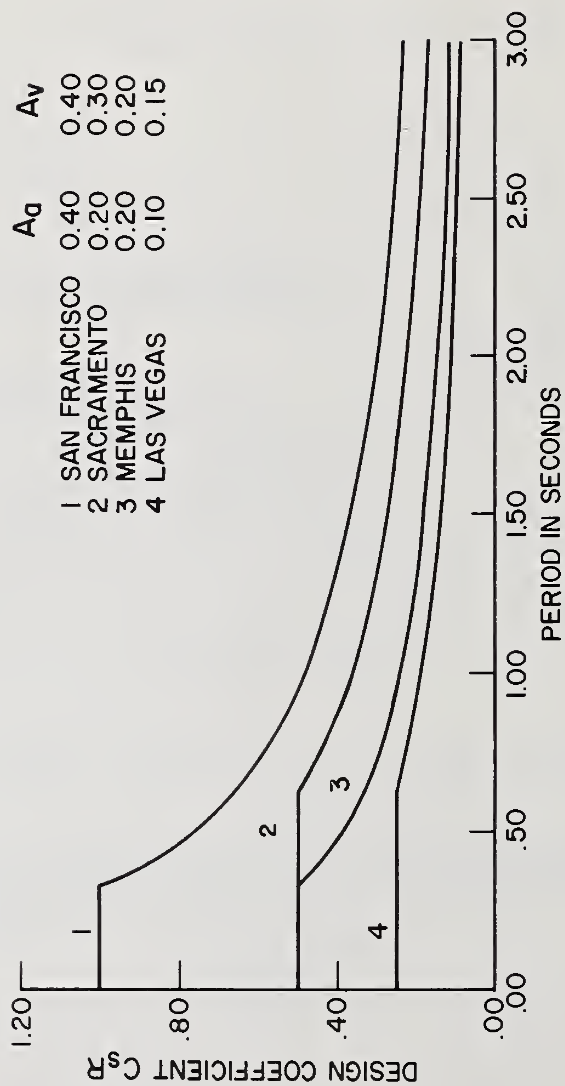
NORMALIZED LATERAL DESIGN FORCE COEFFICIENTS
($A_a = A_v = 1.0$)

FIGURE C1-13



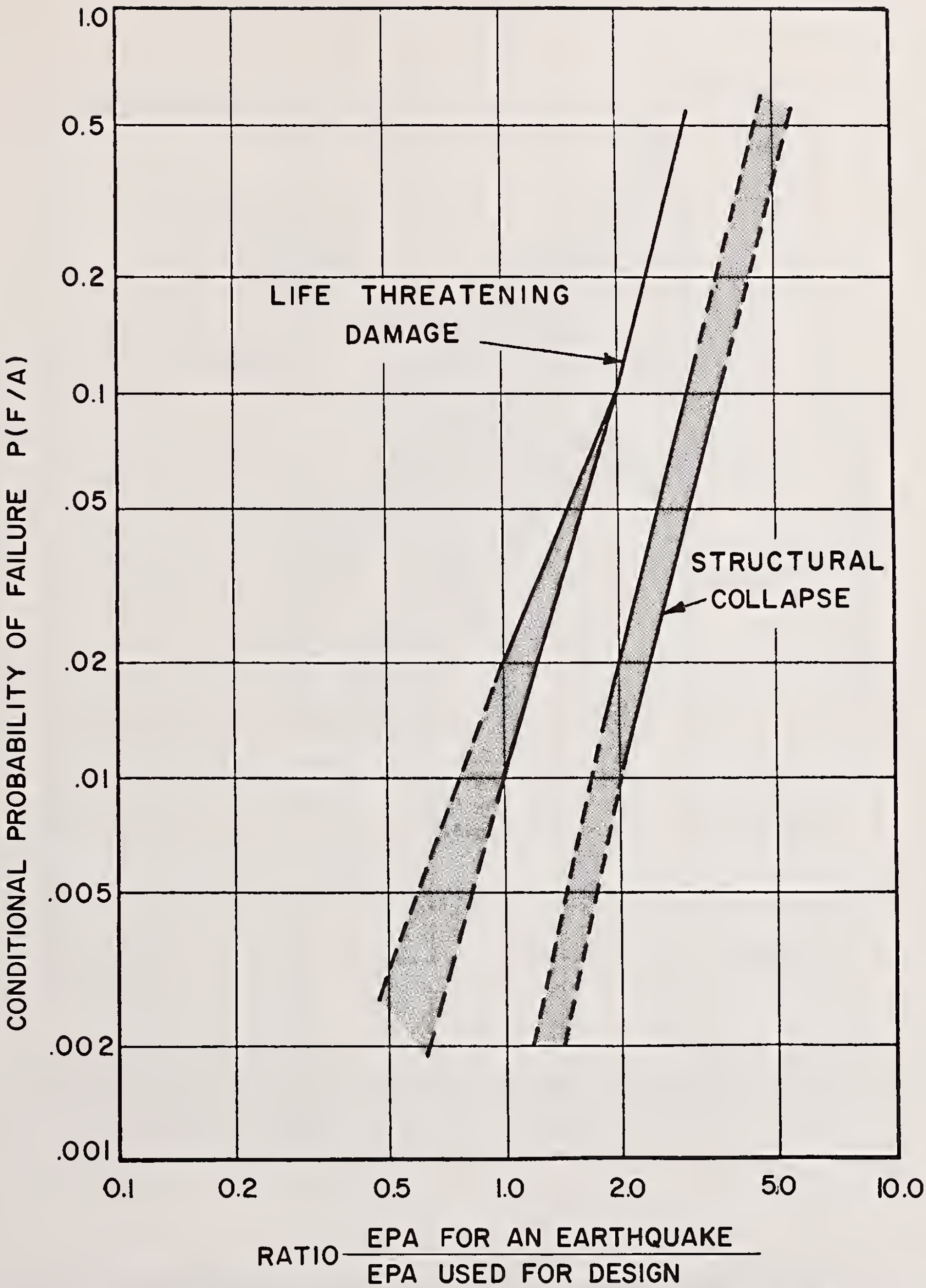
COMPARISON OF FREE FIELD GROUND MOTION SPECTRA
AND LATERAL DESIGN FORCE COEFFICIENTS

FIGURE C1-14



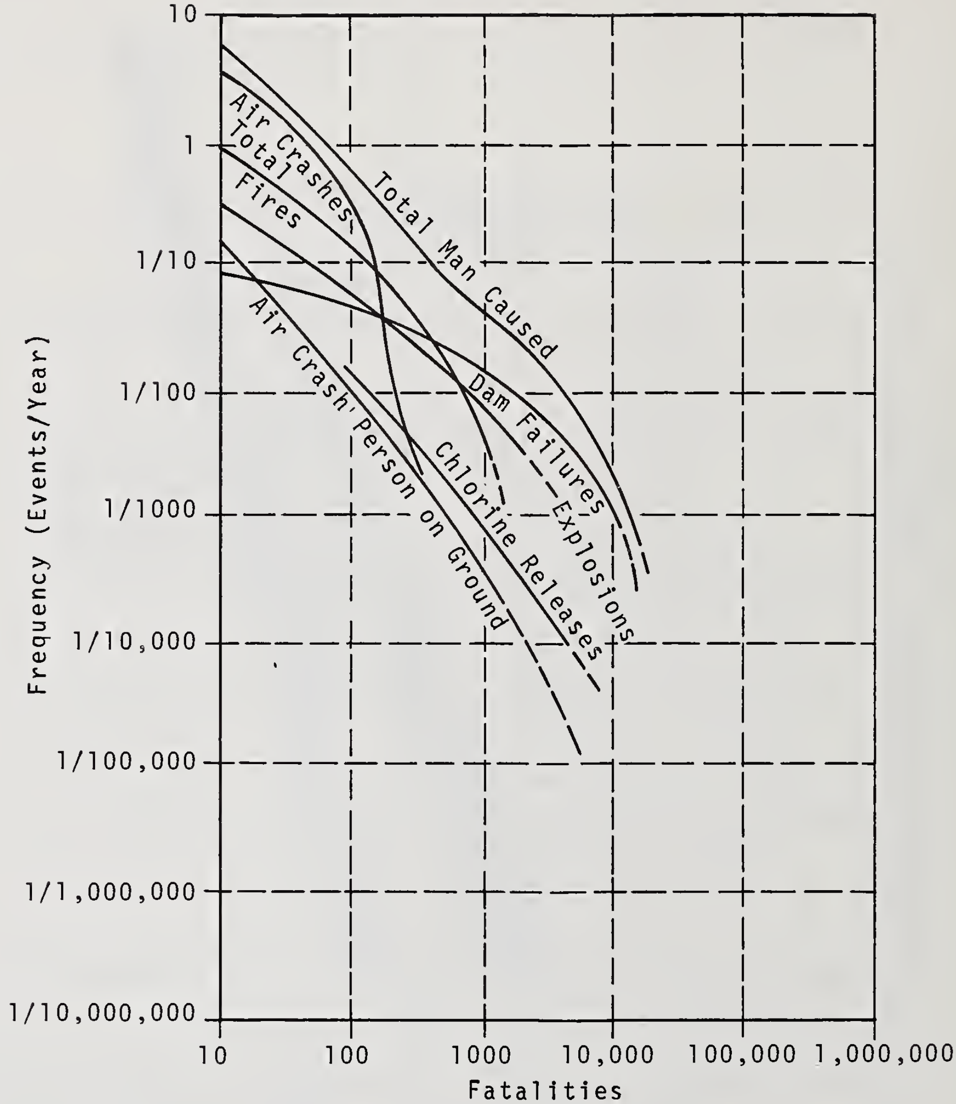
REPRESENTATIVE DESIGN COEFFICIENT CURVES FOR SOIL TYPE S_1 IN FOUR DIFFERENT LOCATIONS

FIGURE C1-15



PROBABILITY OF FAILURE AS A FUNCTION OF ACTUAL EARTHQUAKE RELATIVE TO DESIGN EARTHQUAKE

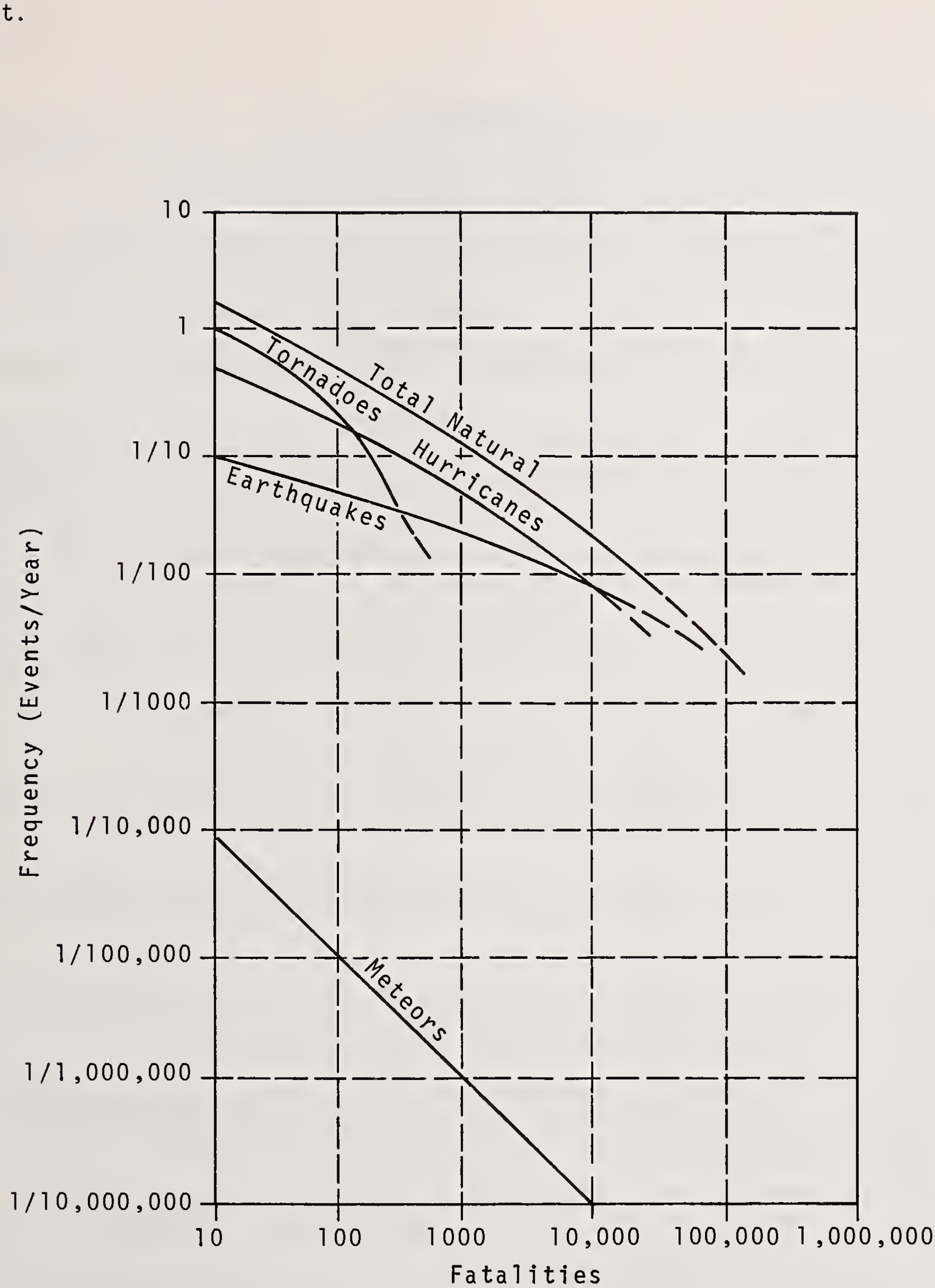
FIGURE C1-16



FATALITIES DUE TO MAN CAUSED FAILURES

Reference: U.S. Nuclear Regulatory Comission (1976) WASH.-1400

FIGURE C1-17



FATALITIES DUE TO NATURAL DISASTERS

Reference: U.S. Nuclear Regulatory Commission (1976)
WASH.-1400

FIGURE C1-18

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COMMENTARY

CHAPTER 3: STRUCTURAL DESIGN REQUIREMENTS

Sec. 3.1 DESIGN BASIS

In these provisions the design of the structure (sizing of individual members, connections, and supports) is based on the internal forces resulting from a linear elastic analysis using the prescribed forces, and assumes that the structure as a whole under these prescribed forces should not deform beyond a point of significant yield. This procedure differs from prior codes wherein the prescribed loads and sizing were at service or working stress levels. Sec. 3.8 prescribes the story drift limits controlling the deformation in the inelastic range when the structure is subjected to the actual seismic forces which may be generated by the specified ground motion.

The term "significant yield" specifically is not the point where first yield occurs in any member but is defined as that level causing complete plastification of at least the most crticial region of the structure, such as formation of the first plastic hinge in the structure. A structural steel frame of compact members is assumed to reach this point when a plastic hinge develops in the most critical member of the structure. A concrete frame reaches this significant yield in its response to the prescribed forces when at least one of the sections of its most critical component reaches its ultimate strength as set forth in Chapter 11. For other structural materials which do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided in their respective material sections (Chapters 9 and 12).

These provisions contemplate a seismic resisting system with redundant characteristics wherein overstrength above the level of significant yield is obtained by plastification at other points in the structure prior to the formation of a complete mechanism.

For example, in the two story bent (Figure C3-1), significant yield is the level where plastification occurs at the most critical joint shown as Joint 1 and as Point 1 on the Load-Deflection diagram. With increased loading, causing the formation of additional plastic hinges, the capacity increases (following the solid line) until a maximum is reached.

The overstrength capacity obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the specified ground motion. The dotted line in Figure C3-1 is the load-deflection curve including the P-delta effects. The dash-dot line is the elasto-plastic curve which results with certain systems and materials.

The response modification factor, R, and the C_d value for deflection amplification (Table 3-B), as well as the criteria for story drift including the P-delta effects, have been established considering that structures generally have additional overstrength capacity, above that whereby the design loads cause significant yield. The R factor essentially represents the ratio of the forces which would develop under the specified ground motion if the structure behaved entirely linearly elastic to the prescribed design forces at significant yield level. This reduction is possible because of the actual energy absorption and energy dissipation capacity (toughness) which the whole structure possesses due to its capability to deform inelastically (the area under the actual load deformation curve). In establishing the R values, consideration has also been given to the performance of the different materials and systems in past earthquakes.

The values of R must be chosen and used with judgment. For example, lower values must be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-delta effects.

It should be noted that the design seismic coefficient (Formula 4-2) does not include a factor which varies for different types of occupancies. This point reflects the belief that increasing the forcing function alone does not necessarily increase the performance and is discussed more fully later in the Commentary. The improved performance characteristics desired for more critical occupancies are provided by the design and detailing requirements set forth in Sec. 3.6 for each Seismic Performance Category and the more stringent drift limits in Table 3-C.

Sec. 3.1 in effect calls for the seismic design to be complete and in accordance with the principles of structural mechanics. The loads must be transferred rationally from their point of origin to the final point of resistance. This should be obvious but is often overlooked by those inexperienced in earthquake engineering.

Sec. 3.2 SITE EFFECTS

The Commentary for Sec. 1.4.1 presents the discussion applicable to Sec. 3.2.1 and 3.2.2. Chapter 6 and Commentary provide background for Sec. 3.2.3, Soil-Structure Interaction.

Sec. 1.4.4 presents site limitations for buildings assigned to Category D. Critical structures needed after a disaster and located in zones of higher seismicity should not be located over an active fault. While it is known that some structures could and must be designed to remain intact even if a fault surface rupture goes through their bases, it is inappropriate for critical facilities to be so located.

Sec. 3.3 FRAMING SYSTEMS

For purposes of these seismic analysis and design provisions, framing systems for buildings are grouped into four general categories of structural systems as shown in Table 3-B. These categories are similar to those contained in the 1974 (and earlier) SEAOC Recommended Lateral Force Requirements and in the 1976 UBC. However, additional breakdown is given for various types of vertical components of the seismic resisting system.

In selecting the structural system, the designer is cautioned to consider carefully the interrelationship between continuity, toughness (including minimizing brittle behavior), and redundancy in the structural framing system, as is subsequently discussed in detail later in this Commentary.

3.3.1 CLASSIFICATION OF FRAMING SYSTEMS

In the selection of the R values for the various systems, consideration was given to the general observed performance of each of the system types during past earthquakes, the general toughness (ability to absorb energy without serious degradation) of the system, and the general amount of damping present in the system when undergoing inelastic response. The designer is cautioned to be especially careful in detailing the more brittle types of systems (low C_d values).

A Bearing Wall System refers to that structural support system wherein major load carrying columns are omitted and the wall and/or partitions are of such strength as to carry the gravity loads (including live loads, floors, roofs, and the weight of the walls themselves). The walls and partitions supply, in plane, lateral stiffness and stability to resist wind and earthquake loading as well as any other lateral loadings. In some cases, vertical trusses are employed to augment lateral stiffness.

In general this system has comparably lower values of R than the other systems due to a frequent lack of providing redundancy for the vertical and horizontal load support. The category designated "light framed walls with shear panels" was intended to cover wood or steel stud wall systems with finishes other than masonry veneers.

A Building Frame System is similar to the "vertical load carrying frame" system described in the SEAOC recommendations. In order to qualify for this system, the gravity loads should be carried primarily by a frame supported on columns rather than bearing walls. Some minor portions of the gravity load can be carried on bearing walls but the amount so carried should not represent more than a few percent of the building area. Lateral resistance is provided by nonbearing structural walls or braced frames.

While there is no requirement in this category to provide lateral resistance in the framing system, it is strongly recommended that some moment resistance be incorporated. In a structural steel frame, this could be in the form of top and bottom clip angles or tees at the beam- or girder-to-column connections. In reinforced concrete, continuity and full anchorage of longitudinal steel and stirrups over the length of beams and girders framing into columns would be a good design practice. With this type of interconnection, the frame provides a nominal secondary line of resistance even though the components of the seismic resisting system are designed to carry all the seismic force.

A Moment Resisting Space Frame System is a system having an essentially complete space frame as in the building frame system. However, in this system, the lateral resistance is provided by moment resisting frames composed of columns with interacting beams or girders. The moment resisting frames may be either Ordinary or Special Moment Frames.

Special Moment Frames shall meet all of the design and detail requirements of Sec. 10.6 or Sec. 11.7 and sections referred to therein. The ductility requirements for these frame systems are required in areas where high seismic hazards are anticipated; see Table 1-A. Where these special design and detailing requirements are not used (in Building Categories A and B), lower R values are specified, indicating that essentially elastic response to earthquake motions is anticipated.

A Dual System consists of a three-dimensional space frame made up of columns and beams which provides primary support for the gravity loads. Lateral resistance is supplied by structural nonbearing walls or bracing; the frame is provided with a redundant lateral force system which is a Special Moment Frame complying with the requirements of Sec. 10.6 and 11.7. The Special Moment Frame is required to be capable of resisting at least 25 percent (judgmentally selected) of the specified seismic force. Normally the Special Moment Frame would be a part of the basic space frame.

The following analyses are required for this category:

1. The frame and shear walls or braced frames shall resist the prescribed lateral seismic force in accordance with the relative rigidities considering fully the interaction of the walls and frames as a single system. This analysis shall be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the frame by the interaction with the shear walls or braced frames shall be considered in this analysis.

2. The Special Moment Frame shall be designed to have a capacity to resist at least 25 percent of the total required lateral seismic force including torsional effects.

Inverted Pendulum Structures were singled out for special consideration because of their unique characteristics and because they are often associated with buildings. Frequently overlooked design aspects and field experience make it desirable to give these structures special attention.

3.3.2 COMBINATIONS OF FRAMING SYSTEMS

For those cases where combinations of structural systems are employed, the designer must use judgment in selecting the appropriate R and C_d values. The intent of Subsec. (A) is to prohibit support of one system by another possessing characteristics which result in a lower base shear factor. The entire system should be designed for the higher seismic shear, as the provision stipulates. The exception is included to permit the use of such systems as a braced frame penthouse on a moment frame building in which the mass of the penthouse does not represent a significant portion of the total building and thus would not materially affect the overall response to earthquake motions.

Subsec. (B) pertains to details and is included to help ensure that the more ductile details inherent with the design for the higher R value system will be employed throughout. The intent is that details common to both systems be designed to remain functional throughout the response in order to preserve the integrity of the seismic resisting system.

3.3.3 through 3.3.5 SEISMIC PERFORMANCE CATEGORIES A, B, C, AND D

General framing system requirements for the four building Seismic Performance Categories A, B, C, and D are given in these subsections. The corresponding design and detailing requirements are given in Sec. 3.6 and Chapters 9 through 12. Any type of building framing system permitted by the provisions may be used for Category A and B buildings. Limitations regarding the use of different structural systems are given for Categories C and D.

Sec. 3.3.4 covers Category C, which compares roughly to the present California design practice for normal buildings other than hospitals. According to the requirements of Chapters 10 and 11, all moment-resisting frames of steel or concrete shall be Special Moment Frames. Note that present SEAOC and UBC recommendations have similar requirements for concrete frames; however, Ordinary Moment Frames of structural steel may be used for heights up to 160 ft (48.6m). In keeping with the philosophy of present codes for zones of high seismic risk, these provisions continue limitations on the use of certain types of structures over 160 ft (48.6 m) in height, but with some changes. Although it is agreed that the lack of reliable data on the behavior of high-rise buildings whose structural systems involve shear walls and/or braced frames makes it convenient at present to establish some limits, the value of 160 ft (48.6 m) as well as that of 240 ft (73.1 m) introduced in these provisions is arbitrary, and considerable disagreement exists among the members of the committee regarding the adequacy of these values. It is recommended that these limitations be the subject of further studies.

(A) These provisions require that buildings over 160 ft (48.6 m) in height shall have one of the following seismic resisting systems:

1. A moment resisting frame system with Special Moment Frames capable of resisting the total prescribed seismic force. This requirement is the same as those of present SEAOC and UBC recommendations.

2. A Dual System as defined in Sec. 2.1, wherein the prescribed forces are resisted by the entire system and the Special Moment Frame is designed to resist at least 25 percent of the prescribed seismic force. This requirement is also similar to the present SEAOC and UBC recommendations. The purpose of

the 25 percent frame is to provide a secondary defense system with higher degrees of redundancy and ductility in order to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. It should be noted that present SEAOC and UBC recommendations require that shear walls or braced frames be able to resist the total required seismic lateral forces independently of the Special Moment Frame. The new provisions require only that the true interaction behavior of the frame-shear wall (or braced frame) system be considered (see Table 3-B). If the analysis of the interacting behavior is based only on the seismic lateral force vertical distribution recommended in the equivalent lateral force procedure of Chapter 4, the interpretation of the results of this analysis for designing the shear walls or braced frame should recognize the effects of higher modes of vibration. The internal forces that can be developed in the shear walls in the upper stories can be more severe than those obtained from such analysis. To avoid analyzing the effects of interaction, the designer can use as a guideline the results of an analysis where the shear walls or braced frames are assumed to resist the total required lateral seismic force acting independently of the Special Moment Frame.

3. The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 ft (73.1 m) is permitted if, and only if, braced frames or shear walls in any plane do not resist more than 33 percent of the seismic design force including torsional effects. The intent of the committee was that each of the shear walls or braced frames be in a different plane and the four or more planes required be spaced adequately throughout the plan or on the perimeter of the building in such a way that the premature failure of one of the single walls or frames would not lead to excessive inelastic torsion.

Although the structural system indicated in Figure C3-2 is acceptable according to the provisions, it is highly recommended that use of such a system be avoided. The intent of the committee is to replace it by the system shown in Figure C3-3. The latter system is believed to be more suitable in view of the lack of reliable data regarding the behavior of tall buildings having structural systems based on central cores formed by coupling slender shear walls or slender braced frames.

Sec. 3.3.5 covers Category D, which is restricted to essential facilities in zones of relatively high seismicity. Because of the necessity for reducing risk (particularly in terms of protecting the life safety or maintaining function by minimizing damage to nonstructural building elements, contents, equipment, and utilities) the height limitations for Category C are reduced. Again, the new limits -- 100 ft (30.5 m) and 160 ft (48.6 m) -- are arbitrary and require further study. The committee believes that, at present, it is advisable to establish these limits, but the importance of having more stringent requirements for detailing the seismic resisting system as well as the nonstructural components of the building must be stressed. Such requirements are specified in Sec. 3.6 and 3.7 and Chapters 9 through 12.

Sec. 3.4 BUILDING CONFIGURATION

The configuration of a building can significantly affect its performance during a strong earthquake which produces the ground motion contemplated in these provisions. Configuration can be divided into two aspects, plan configuration and vertical configuration. The provisions were basically derived for buildings having regular configurations. Past earthquakes have repeatedly shown that buildings having irregular configurations suffer greater damage than buildings having regular configurations. This situation prevails even with good design and construction. These provisions are designed to encourage that buildings be designed to have regular configurations.

Sec. 3.4.1 specifies plan configuration requirements. A building having a regular configuration could be square or rectangular or circular. A square or rectangular building with minor re-entrant corners would still be considered regular but large re-entrant corners creating a crucifix form would be classified as an irregular configuration. The response of the wings of this type of building is generally different than the response of the building as a whole, and this produces higher local forces than would be determined by application of these provisions without modification. Other plan configurations such as H-shapes which have a geometrical symmetry would also be classified as irregular because of the response of the wings.

A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass or vertical seismic resisting elements. Torsional effects in earthquakes can occur even when the static centers of mass and resistance coincide, and these effects can magnify the torsion due to eccentricity between the static centers. For this reason, buildings having an eccentricity between the static center of mass and the static center of resistance in excess of 10 percent of the building dimension perpendicular to the direction of the seismic force should be classified as irregular. The vertical resisting components may be arranged so the static centers of mass and resistance are within the limitations given above and still be unsymmetrically arranged so that the prescribed torsional forces would be unequally distributed to the various components.

There is a second type of distribution of vertical resisting components which, while not being classified as irregular, does not perform well in strong earthquakes. This arrangement is termed a core-type building with the vertical components of the seismic resisting system concentrated near the center of the building. Better performance has been observed when the vertical components are distributed near the perimeter of the building.

Significant differences in stiffness between portions of a diaphragm at a level are classified as irregularities since these may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular building. Examples of plan irregularities are illustrated in Figure C3-4.

Sec. 3.4.2 covers vertical configuration. Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels which are significantly different from the distribution assumed in the equivalent lateral force procedure given in Chapter 4. One type of vertical irregularity is created by unsymmetrical geometry with respect to the vertical axis of the building. The building may have a geometry which is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offsets at one or more levels. An offset would be considered significant when the ratio of the smaller dimension to the larger dimension is less than 75 percent. The building would also be considered irregular if the smaller dimension were below the larger dimension, creating an inverted pyramid effect.

A building would be classified as irregular where the ratio of mass to stiffness in adjoining stories differs significantly. This might occur when a heavy mass, such as a swimming pool, was placed at one level. A moment resisting frame building might be classified as having a vertical irregularity if one story were much taller than the adjoining stories and the resulting decrease in stiffness which would normally occur was not, or could not be, compensated for. Examples of vertical irregularities are illustrated in Figure C3-5.

Sec. 3.5 ANALYSIS PROCEDURES

Many of the standard procedures for the analysis of forces and deformations in buildings subjected to earthquake ground motion, including the two procedures specified in these design provisions, are listed below in order of increasing rigor and expected accuracy.

1. Equivalent Lateral Force Procedure (Chapter 4).

2. Modal Analysis Procedure with one degree of freedom per floor in the direction being considered (Chapter 5).

3. Modal Analysis Procedure with several degrees of freedom per floor.

4. Inelastic Response History Analysis: step-by-step integration of the coupled equations of motion with one degree of freedom per floor in the direction being considered.

5. Inelastic Response History Analysis: step-by-step integration of the coupled equations of motion with several degrees of freedom per floor.

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as specified in Chapter 6 or through a more complete analysis of this interaction as appropriate. Every procedure improves in rigor if combined with use of results from experimental research (not described in these design provisions).

The Equivalent Lateral Force (ELF) procedure specified in Chapter 4 is similar in its basic concept to the past SEAOC recommendations (1-3)*, but several improved features have been incorporated.

The modal superposition method (4-7)* is a general procedure for linear analysis of the dynamic response of structures. In various forms, modal analysis has been widely used in the earthquake-resistant design of special structures such as very tall buildings, offshore drilling platforms, dams, and nuclear power plants, but this is the first time that modal analysis has been included in design provisions for buildings. The Modal Analysis Procedure specified in Chapter 5 is simplified from the general case by restricting consideration to lateral motion in a plane. Only one degree of freedom is required per floor for this type of motion.

The ELF procedure of Chapter 4 and the Modal Analysis procedure specified in Chapter 5 are both based on the approximation that the effects of yielding can be adequately accounted for by linear analysis of the seismic resisting system for the design spectrum, which is the elastic acceleration response spectrum reduced by the response modification factor, R. The effects of (1) the horizontal component of ground motion perpendicular to the direction under consideration in the analysis, (2) the vertical component of ground motion, and (3) torsional motions of the structure are all considered in the same simplified approaches in the two procedures. The main difference between the two procedures lies in the distribution of the seismic lateral forces over the height of the building. In the Modal Analysis Procedure, the distribution is based on properties of the natural vibration modes, which are determined from the actual mass and stiffness distribution over the height. In the ELF procedure, the distribution is based on simplified formulas which are appropriate for regular buildings as specified in Sec. 3.4 and 3.5. Otherwise the two procedures are subject to the same limitations.

The two analytical procedures are likely to be inadequate if the lateral motions in two orthogonal directions and the torsional motion are strongly coupled. Such would be the case if the building were irregular in its plan configuration (see Sec. 3.4), or if it had a regular plan but its lower natural frequencies were nearly equal and the centers of mass and resistance were nearly coincident. A general model for the analysis of such buildings would include at least three degrees of freedom per floor, two translational motions, and one torsional. Such a structure would usually have many modes which show a

*See References at end of Chapter 5 Commentary.

combination of translational and torsional motion. Analysis procedures similar to those specified in Chapter 5 can be applied to buildings of this type, with suitable generalization of the concepts involved. It is necessary, for example, to account for the facts that a given mode might be excited by both horizontal components of ground motion, and modes which are primarily torsional can be excited by the translational components of the ground shaking.

The methods of modal analysis can be generalized further to model the effect of diaphragm flexibility, soil-structure interaction, etc. In the most general form, the idealization would take the form of a large number of mass points, each with six degrees of freedom (three translation and three rotational) connected by generalized stiffness elements.

The ELF procedure (Chapter 4) and both versions of the Modal Analysis Procedure, (the simple version given in Chapter 5 and the general version with several degrees of freedom per floor mentioned in the foregoing paragraphs) are all likely to err systematically on the unsafe side if story strengths are distributed irregularly over height. This feature is likely to lead to a concentration of ductility demand in a few stories of the building. A simple procedure to account for irregular strength distribution is discussed in this Commentary (Sec. 3.7.3).

The actual strength properties of the various components of a building can be explicitly considered only by a nonlinear analysis of dynamic response by direct integration of the coupled equations of motion. This method has been used extensively in research studies of earthquake response of yielding structures. If the two lateral motions and the torsional motion are expected to be essentially uncoupled, it would be sufficient to include only one degree of freedom per floor, the motion in the direction along which the building is being analyzed; otherwise at least three degrees of freedom per floor, two translational motions and one torsional, should be included. It should be recognized that results of nonlinear response history analysis of such mathematical building models are only as good as are the models chosen to represent the building vibrating at large amplitudes of motion, large enough to cause significant yielding during strong ground motions. Furthermore, reliable results can be achieved only by calculating the response to several ground motions -- recorded accelerograms and/or simulated motions -- and examining the statistics of response.

The least rigorous analytical procedure which may be used in determining the design earthquake forces and deformations in buildings depends on three factors: Seismicity Index; Seismic Performance Category; and structural characteristics (in particular, regularity). Regularity is defined in Sec. 3.4.

If a building is classified as Seismic Hazard Exposure Group III in Seismicity Index 1, its failure could be significant to the public safety. For all other regular buildings in higher index areas, it is required that the ELF procedure in Chapter 4 be used, except that a more rigorous procedure may be required for some buildings in areas having Seismicity Indices 3 and 4.

The basis for the ELF procedure and its limitations were discussed in prior paragraphs of this Commentary. The ELF procedure is adequate for most regular buildings. The designer may wish to employ a more rigorous procedure (see list of procedures at beginning of Sec 3.5. of this Commentary) for those regular buildings where it may be inadequate; some of these situations have been mentioned earlier.

The ELF procedure is likely to be inadequate in the following cases: buildings with irregular mass and stiffness properties in which case the simple formulas for vertical distribution of lateral forces (Formulas 4-6 and 4-6a) may lead to erroneous results; buildings (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strongly coupled; and buildings with irregular distribution of story strengths leading to possible concentration of ductility demand in a few stories of the building. In such cases, a more rigorous procedure which considers the dynamic behavior of the structure should be employed. Such special consideration is necessary only for irregular buildings (see Sec. 3.4) which are located in areas with high seismicity (those associated with Seismicity Indices 3 and 4) and whose failure would pose significant hazard to the public, those housing Seismic Hazard Exposure Groups II and III. The preceding discussion of the capabilities and limitations of the various analytical procedures should be helpful to the designer in selecting a suitable analytical procedure.

Buildings in Categories B, C, and D with certain types of vertical irregularities may be analyzed as regular buildings in accordance with the provisions of Chapter 4. These buildings are generally referred to as setback buildings. The procedure delineated below may be used.

1. The base and tower portions of a building having a setback vertical configuration may be analyzed as indicated in (2) below if all of the following conditions are met:

   a. The base portion and the tower portion, considered as separate buildings, can be classified as regular.

   b. The stiffness of the top story of the base is at least five times that of the first story of the tower.

   Where these conditions are not met, the building shall be analyzed in accordance with Chapter 5.

2. The base and tower portions may be analyzed as separate buildings in accordance with the following:

   a. The tower may be analyzed in accordance with the procedures in Chapter 4 with the base taken at the top of the base portion.

   b. The base portion shall then be analyzed in accordance with the procedures in Chapter 4 using the height of the base portion of h_n and with the gravity load and base shear of the tower portion acting at the top level of the base portion.

The design provisions in Chapter 5 include a simplified version of modal analysis which accounts for irregularity in mass and stiffness distribution over the height of the building. It would be adequate, in general, to use the ELF procedure for buildings whose seismic resisting system has the same configuration in all stories and in all floors, and whose floor masses and cross-sectional areas and moments of inertia of structural members do not differ by more than 30 percent in adjacent floors and in adjacent stories. For other buildings, the following criteria should be applied to decide whether the modal analysis procedures of Chapter 5 should be used. The story shears should be computed using the ELF procedure specified in Chapter 4. On this basis, structural members should be approximately dimensioned. The lateral displacements of the floor can then be computed. Replacing h_x^k in Formula 4-6a with these displacements, one recomputes lateral forces, and from these new story shears are obtained. If at any story the recomputed story shear differs from the corresponding value as obtained from the procedures of Chapter 4 by more than 30 percent, the building should be analyzed using the procedure of Chapter 5. If the difference is less than this value, the building may be designed using the story shear obtained in the application of the present criterion and the procedures of Chapter 5 are not required.

Application of the present criterion to these buildings requires far less computational effort than the use of the Modal Analysis Procedure of Chapter 5, and in the majority of the buildings, use of the criterion will determine that the latter need not be used; at the same time, the present criterion furnishes a set of story shears which practically always lie much closer to the results of modal analysis than the results of the ELF procedure.

This criterion is equivalent to a single cycle of Newmark's method for calculation of the fundamental mode of vibration. The criterion is such that it will detect both unusual shapes of the fundamental mode and excessively high influence of higher modes. Numerical studies have demonstrated that this criterion for determining whether modal analysis must be used will, in general, detect cases which truly should be analyzed dynamically; it will not, in general, indicate the need for dynamic analysis when its application would not greatly improve accuracy.

Sec. 3.6 DESIGN AND DETAILING REQUIREMENTS

The design and detailing requirements for components of the seismic resisting system are stated in this section. General detailing requirements are specified in Sec. 3.7. Some of the requirements introduced by these provisions are not found in present code provisions; all of the requirements cited are spelled out in considerably more detail, and in most cases are more stringent than existing provisions. The main reasons for this follow.

The provision of detailed design ground motions and requirements for analysis of the structure do not by themselves make a building earthquake resistant. Additional design requirements are necessary to provide a consistent degree of earthquake resistance in buildings. The more severe the expected seismic ground motions, the more stringent these additional design requirements should be. Not all of the necessary design requirements are expressed in codes, and while experienced seismic design engineers account for them, they are often overlooked by engineers lacking experience in design and construction of earthquake-resistant structures.

Considerable uncertainties exist regarding (1) the actual dynamic characteristics of future earthquake motions expected at a building site; (2) the soil-structure foundation interaction; (3) the actual response of buildings when subjected to seismic motions at their foundations; and (4) the mechanical characteristics of the different structural materials, particularly when they undergo significant cyclic straining in the inelastic range which can lead to severe reversals of strains. It should be noted that the overall inelastic response of a structure is very sensitive to the inelastic behavior of its critical regions, and this behavior is influenced, in turn, by the detailing of these regions.

Although it is possible to counteract the consequences of these uncertainties by increasing the level of design forces, it was considered more feasible to provide a building system with the largest energy dissipation consistent with the maximum tolerable deformations of nonstructural components and equipment. This energy dissipation capacity, which is usually denoted simplistically as "ductility," is extremely sensitive to the detailing. Therefore, in order to achieve such a large energy dissipation capacity, it is essential that stringent design code requirements be used for detailing the structural as well as nonstructural components and their connections or separations. Furthermore, it is necessary to have good quality control of materials and competent inspection. The importance of these factors has been clearly demonstrated by the building damage observed after moderate and severe earthquakes. It should be kept in mind that a building's response to seismic ground motion most often does not reflect the designer's or analyst's original conception or modeling of the structure on paper. What is reflected is the manner in which the building was constructed in the field. These provisions emphasize the importance of detailing and recognize that the detailing requirements should be related to the expected earthquake intensities and the importance of the building's function and/or the density and type of occupancy. The greater the expected intensity of earthquake ground shaking (Seismicity Index) and the more important the function of or number of occupants in the building, the more stringent the design and detailing requirements should be. In defining these requirements, the provisions have introduced the concept of Seismic Performance Categories (Table 1-A). These relate to the Seismicity Index (Sec. 1.4.1) and the Seismic Hazard Exposure Group (Sec. 1.4.2).

3.6.1 SEISMIC PERFORMANCE CATEGORY A

Because of the very low seismicity associated with Seismicity Index 1, it is considered appropriate for Category A buildings to only require good quality of construction materials and adequate ties and anchorage, as specified in Sec. 3.7.5, 3.7.6, 3.7.7, and 7.3. Category A buildings would be constructed in the major portions of the United States (US) which are low earthquake risk areas, but most of which is subject to strong winds. Those promulgating construction regulations for these areas might give consideration to many of the low-level seismic provisions as being suitable to reduce windstorm hazard. The provisions consider only earthquakes and therefore no other requirements are prescribed for Category A buildings. Only wind design in accordance with the local code is required, with the added requirements of ties and wall anchorage added by these provisions.

In low earthquake risk areas, it is unrealistic to believe that construction practices will change overnight. However, if existing requirements can be improved gradually, a major reduction in potential hazard can be achieved at low cost and with little inconvenience.

3.6.2 SEISMIC PERFORMANCE CATEGORY B

Areas where Category B buildings would be constructed comprise the next largest portion of the US. Appreciable increases in earthquake-resistant requirements are specified as compared to Category A, but are quite simplified as compared to present requirements in areas of high seismicity.

The material requirements in Chapters 9 through 12 for Category B are somewhat more restrictive than those for Category A. Unreinforced masonry can be used nonstructurally; but if masonry is used as part of the lateral force resisting system, it must be partially reinforced for buildings up to 35 ft in height and fully reinforced in buildings over 35 ft in height.

Concrete frames must be semiductile with some transverse reinforcement in the joint. Steel frames must be elastic, i.e., similar to ductile or plastic design requirements with compact section requirements somewhat relaxed. Wood framing has certain minimal restrictions on diaphragms, lag screws, etc. These are discussed in the commentary for Chapters 9 through 12.

The general Category B requirements specifically recognize the need to design diaphragms, provide collector bars, and provide reinforcing around openings. These requirements may seem elementary and obvious, but because these requirements are not specifically required in current codes, many engineers totally neglect them. A nominal interconnection between pile caps and caissons is also required.

3.6.3. SEISMIC PERFORMANCE CATEGORY C

Category C requirements compare roughly to present design practice in California seismic areas for buildings other than schools and hospitals. All masonry must be reinforced. All moment resisting frames of concrete or steel must meet ductility requirements. Building separations to prevent pounding, interaction effects between structural and nonstructural elements, and effects of lateral force deformations on vertical load capacity (P-delta) must be investigated. Foundation interaction requirements are increased. Wood design requirements are similar to those of the 1976 UBC.

Experience in past earthquakes has demonstrated that unreinforced masonry or unreinforced concrete performs poorly and is hazardous even when used in nonstructural elements. Consequently, all concrete and masonry construction must be reinforced for Category C construction.

Moment resisting space frames can be classified into two levels of ductility: Ordinary and Special Moment Frames. Each type has its own R and C_d coefficient. Above 160 ft (48.6 m) in height, only Special Moment Frames can be used either with or without shear walls or braced frames with appropriate R values. For a redundant system when the bracing is provided with shear walls or braced frames, the height limit is extended to 240 ft (73.1 m).

In frame buildings under 160 ft (48.6 m) in height, a more rigid system with a lower R value may be used as a support. The extra elastic strength of the more rigid system should reduce the possibility of yield in the critical lower stories.

The response of a building will depend not only on the structural elements which the designer has calculated, but on all elements, structural and nonstructural, calculated or not. In the initial stages of a large earthquake, the base shear and the distribution of shear throughout the height of a building, for example, will be distributed to both structural and nonstructural elements strictly in accordance with their effective rigidities. In essence, rigid elements which are physically divorced from the structure by flexible connections will not be reliably effective for resisting shears. However, some stiffness due to friction or the force necessary to cause the connections to bend will contribute to the shortening of the building period.

The enclosing of the space frame by rigid nonstructural components materially changes the distribution of the internal forces of the structure. For example, if a nonstructural, fairly strong partition is rigidly attached to a moment resisting frame, that frame bent will act as a shear wall until failure of the partition occurs. As a shear wall, it will resist more load than the designer assumed, with higher overturning stresses, different diaphragm shears, etc. In some earthquakes, this uncalculated redistribution of forces has caused structural components to fail before the nonstructural partitions failed.

Formula 4-3 (for period) in Sec. 4.2 partially accounts for this stiffening effect, since it is based on observations of actual buildings before, during, and after earthquakes. Any stiffening effect in the building due to nonstructural components must be accounted for in the period determination of the structure and consequently in the design.

In many buildings, the seismic resisting sysem 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 resisting frames. Ordinarily the internal slabs and columns which resist gravity loads but not lateral seismic loads are not designed to resist seismic loads since their resistance is small in comparison with the resistance of the exterior walls or frames. However, although they are not needed for lateral resistance, they do deform with the rest of the structure as it deforms under lateral loads. Subsec. (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 noted above, there will be bending moments in the columns and slabs and an uneven shear distribution at the column capitals. At the calculated deflections (using C_d as noted elsewhere) and the resulting imposed moments and shears, it must be demonstrated that the members and connections will not fail under the design gravity loadings. The loading is cyclical, so static ultimate load capacities are not acceptable. If the combination of these loads and deformations results in stresses below yield, it can be assumed that the system is capable of supporting the gravity loads. If the stresses are above yield, then sufficient ductility under cyclic loading must be provided. If the gravity load bearing system is to provide any calculated resistance to the seismic resisting system (no matter how small), then the detailing for ductility must be consistent with the values given in Table 3-B.

3.6.4 SEISMIC PERFORMANCE CATEGORY D

Category D construction is required for critical structures in relatively high seismic zones. It is deemed prudent that these structures not be located over the trace of an active fault which could cause ground rupture (see Sec. 1.4.4). Because of the necessity for reduced risk, the height limitations are reduced (see Sec. 3.3.5). The specific material provisions include additional requirements and limitations for the design of this building category.

Sec. 3.7 STRUCTURAL COMPONENT LOAD EFFECTS

This Section specifies that the direction of the applied seismic force shall be that which produces the most critical load effect on the building. In past codes, it was only necessary to independently consider loads on the main axes of the building. For beams and girders, this gives maximum design stresses. However, if the earthquake forces affect the building in a direction other than the main axes, it can be shown that the corner columns are subjected to higher stresses. This may be a partial explanation of the vulnerability of such columns in past earthquakes. Sec. 3.7.2 requires that the effects from seismic loads applied in one direction be combined with those from the other direction. Due to possible out-of-phase effects, 30 percent of the minor load, not 41 percent, must be added to the major axis. In some cases where there are major torsional effects or various types and directions of framing, this may affect more than just the columns. Attention is also called in this paragraph to the necessity for considering possible detrimental P-delta effects.

3.7.1 COMBINATION OF LOAD EFFECTS

The committee reviewed various combination of load effects formulas and other data before arriving at Formulas 3-1, 3-2, and 3-2a. For example, since 1956 the American Concrete Institute has based design on the cross-sectional strength of component members. They have included load combinations which are believed to be consistent with the strength reduction factors (based in part on considerations of statistical variability of properties) to produce a margin of safety for most design loading which is generally acceptable to the design professions. No specific study was made for earthquake loading, and the load combinations were set to be compatible with previous working stress load combinations.

A subcommittee within ANSI Committee A58.1 is currently studying the problem with the stated aim of arriving at a compatible combination of load effects for all building system materials. No results of their study are available. After carefully evaluating the available material and past experience and exercising reasonable engineering judgment, the committee decided to express the load effect combinations involving seismic design in a format similar to that used in ACI 318 but with the values changed for the following reasons:

1. The basic load factor used in ACI 318 to account for variability of dead load effects is 0.75 x 1.4 = 1.05 (the 1.4 was 1.5 before 1971). This factor combines with the appropriate understrength factor to produce a design that is judged adequate on the basis of the ultimate strength of individual members. On an average, actual dead loads have been found to be 5 to 10 percent larger than those calculated in design. Thus it is reasonable to use a factor of 1.05 on dead load in seismic design.

In Formulas 3-1 and 3-2, a factor of ± 20 percent was placed on the dead load to account for the effects of vertical acceleration. The concurrent maximum response of vertical accelerations and horizontal accelerations, direct and orthogonal, is unlikely and therefore the direct addition of responses was not considered appropriate. For elements in which tensile mode of failure is relatively brittle, a more conservative factor of 50 percent on the dead load was chosen for Formula 3-2a.

A study was made to see if these factors could be modified to give consideration to the different seismic areas. The resulting complexity of the load combination formulas could not be justified. Thus the decision was made to keep one set of formulas for all areas.

2. The live load factor of ACI 318 is 0.75 x 1.7 = 1.3. This factor was chosen in order to simplify the load combination determinations since the 0.75 factor appears in both dead and live load. The terms "maximum lifetime live load" and "instantaneous live load" are used. The maximum lifetime live load is assumed to be represented by the code-specified live loads. In most instances, the actual instantaneous live load is very much smaller than the maximum lifetime live load, which acts for a short time period and is generally applied to a small portion of the structure. For the purpose of the ATC provisions, it was decided to use only the code-specified loads for the present. A load factor of 1.0 was chosen to partially recognize the lower values for the instantaneous live load for combination with earthquake load effects.

3. For a combination with the design earthquake, it is assumed that an instantaneous snow load for combination with earthquake loads is the same as that expressed in the 1976 UBC.

4. The design basis expressed in Sec. 3.1 reflects the fact that the specified earthquake loads are at the design level without amplification by load factors; thus the load factor of 1.0 is assigned to the earthquake load effects in Formulas 3-1, 3-2, and 3-2a.

Sec. 3.7.2 ORTHOGONAL EFFECTS

Earthquake forces act in both principal directions of the building simultaneously, but the earthquake effects in the two principal directions are unlikely to reach their maximum simultaneously. This section provides a reasonable and adequate method of combining them. It requires that structural elements be designed for 100 percent of the effects of seismic forces in one principal direction combined with 30 percent of the effects of seismic forces in the orthogonal direction. The following combinations of effects of gravity loads, effects of seismic forces in the x-direction, and effects of seismic forces in the y-direction (orthogonal to x-direction) thus pertain:

gravity ±100% of x-direction ± 30% of y-direction
gravity ± 30% of x-direction ±100% of y-direction

The combination and signs (plus or minus) requiring the greater member strength are used for each member. Orthogonal effects are slight on beams, girders, slabs and other horizontal elements that are essentially one-directional in their behavior, but they may be significant in columns or other vertical members which participate in resisting earthquake forces in both principal directions of the building.

3.7.3 DISCONTINUITIES IN STRENGTH OF VERTICAL RESISTING SYSTEM

This Section requires consideration of discontinuities in strength. It is not generally recognized that large discontinuities in story strength can cause adverse response effects in a building. Usual practice is to determine what size, length, or strength of a resisting element is required; if more than the required strength is provided, so much the better. Unfortunately the extra strength in a story, if significantly different than that in adjacent stories, can produce responses which vary greatly from those calculated by using the procedures in Chapter 4 or 5.

The committee considered the following approach to this problem:

1. Compute the ratio of shear capacity to the design shear for each story. Denote this ratio for story n by r_n.
2. Compute r, the average of r_n over all stories.
3. If for any story r_n is less than (2/3)r, modify R and C_d for the building as given by Table 3-B to $\tilde{R}$ and $\tilde{C}_d$

where

$$\tilde{C}_d = 1 + \frac{C_d - 1}{2}$$

$$\tilde{R} = \frac{\tilde{C}_d}{C_d} R$$

4. Use $\tilde{R}$ instead of R to recompute the lateral forces, and $\tilde{C}_d$ instead of C_d in computing story drifts.

The committee feels that further study should be given to this problem.

3.7.4 NONREDUNDANT SYSTEMS

Consideration should be given in the design to potentially adverse effects where there is a lack of redundancy. Because of the many unknowns and uncertainties in the magnitude and characteristics of the earthquake loading, in the materials and systems of construction for resisting earthquake loadings, and in the methods of analysis, good earthquake engineering practice has been to provide as much redundancy as possible in the seismic resisting system of buildings.

Redundancy plays an important role in determining the ability of the building to resist earthquake forces. In a structural system without redundant components, every component must remain operative to preserve the integrity of the building structure. On the other hand, in a highly redundant system, one or more redundant components may fail and still leave a structural system which retains its integrity and can continue to resist lateral forces, albeit with diminished effectiveness.

Redundancy is often accomplished by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic resisting system. These multiple points of resistance can prevent a catastrophic collapse due to distress or failure of a member or joint. The overstrength characteristics of this type of frame are also discussed earlier in the commentary in Sec. 3.1.

Redundant characteristics can also be obtained by providing several different types of seismic resisting systems in a building. The backup system can prevent catastrophic effects if distress occurs in the primary system.

In summary, it is good practice to incorporate redundancy into the seismic resisting system and not to rely on any system wherein distress in any member may cause progressive or catastrophic collapse.

3.7.5 TIES AND CONTINUITY

The analysis of a structure and the provision of a design ground motion alone do not make a structure earthquake resistant; additional design requirements are necessary to provide adequate earthquake resistance in buildings. While experienced seismic designers normally provide them, some of the requirements have not been previously formally required and consequently they have often been overlooked by inexperienced engineers.

Probably the most important single attribute of an earthquake-resistant building is that it is tied together to act as a unit, but no previous code has stated this requirement. This attribute is not only important in earthquake resistant design, but is indispensable in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. Sec. 3.7.5 requires that all parts of the building (or unit if there are separation joints) be so tied together that any section passed through any part of the structure is tied to the rest for a force of $A_v/3$ with a minimum of 5 percent g. In addition, beams must be tied together and beams tied to their supports or columns and columns to footings for a minimum of 5 percent of the dead and live load reaction.

3.7.6 CONCRETE OR MASONRY WALL ANCHORAGE

One of the major hazards from buildings during an earthquake is the pulling away of heavy masonry or concrete walls from the floors or roofs. While requirements for the anchorage to prevent this separation have been common in highly seismic areas, they have been minimal or nonexistent in most parts of the country. The requirement has been added in this section that anchorage will be required in any locality to the extent of 1,000 A_v lbs/lin. ft. While this requirement of itself may not provide complete earthquake-resistant design, observations of earthquake damage indicate that this provision can greatly increase the earthquake resistance of buildings and reduce hazards in those localities where earthquakes may occur but are rarely damaging.

In addition to the above general requirements, additional requirements related to the expected earthquake intensities and the occupancy of the structure are imposed in various zones. To accommodate and define these requirements, the concept of Seismic Performance Category was introduced in Sec. 1.4. The Seismicity Index and the Seismic Hazard Exposure Group (occupancy or function of the building) are used in assigning buildings to Seismic Performance Categories (Sec. 1.4 and Table 1-A).

3.7.7 ANCHORAGE OF NONSTRUCTURAL SYSTEMS

Anchorage of nonstructural systems and components of buildings is required when prescribed in Chapter 8.

3.7.8 COLLECTOR ELEMENTS

Many buildings in ordinary practice have shear walls or other bracing elements which are not uniformly spaced around the diaphragms. Such conditions require that collector or drag bars be provided. A simple illustration is shown in Figure C3-6. Consider a building as shown in the plan with four short shear walls at the corners arranged as shown. For north-south earthquake forces, the diaphragm shears on line AB are uniformly distributed between A and B, if the chord reinforcing is assumed to act on lines BC and AD. However, wall A is quite short, so reinforcing steel is required to collect these shears and transfer them to the wall. If wall A is a quarter of the length of AB, the steel must carry, as a minimum, three-fourths of the total shear on line AB. The same principle is true for the other walls. In Figure C3-7, reinforcing is required to collect the shears or drag the forces from the diaphragm into the shear wall. Similar collector elements are needed in most shear walls and some frames.

3.7.9 DIAPHRAGMS

Diaphragms are deep beams or trusses which distribute the lateral loads from their origin to the components where they are resisted. As such they are subject to shears, bending moments, direct stresses (truss member, collector elements), and deformations. The deformations must be minimized in some cases because they could overstress the walls to which they are connected. The amount of deflection permitted in the diaphragm must be related to the ability of the walls (normal to the direction being analyzed) to deflect without failure.

A detail which is commonly overlooked by many engineers is the requirement to tie the diaphragm together so that it acts as a unit. Wall anchorages tend to tear off the edges of the diaphragm; thus the ties must be extended into the diaphragm so as to develop adequate anchorage. In several industrial buildings during the San Fernando earthquake, seismic forces from the walls caused separations in the roof diaphragm twenty or more feet from the edge.

When openings occur in shear walls, diaphragms, etc., it is not adequate to only provide temperature trimbars. The chord stresses must be provided for and the chords anchored to develop the chord stresses by embedment. The embedment must be sufficient to take the reactions without overstressing the material in any respect. Since the design basis depends on an elastic analysis, the internal force system should be compatible with both statics and the elastic deformations.

3.7.10 BEARING WALLS

A minimum anchorage of bearing walls to diaphragms or other resisting elements is specified. To ensure that the walls and supporting framing system interact properly, it is required that the interconnection of dependent wall elements and connections to the framing system have sufficient ductility or rotational capacity, or strength, to stay as a unit. Large shrinkage or settlement cracks can significantly affect the desired interaction.

3.7.11 INVERTED PENDULUM-TYPE STRUCTURES

Inverted pendulum-type structures have a large portion of their mass concentrated near the top, and thus have essentially one degree of freedom in horizontal translation. Often the structures are T-shaped with a single column supporting a beam or slab at the top. For such a structure, the lateral motion is accompanied by rotation of the horizontal element of the T due to rotation at the top of the column, resulting in vertical accelerations acting in opposite directions on the overhangs of the structure. Hence a bending moment would be induced at the top of the column although the procedures of Sec. 4.2 and 4.5 would not so indicate. A simple provision to compensate for this is specified in this section. The bending moments due to the lateral force are first calculated for the base of the column according to the provisions of Sec. 4.2 and 4.5. One-half of the calculated bending moment at the base is applied at the top and the moments along the column are varied from 1.5 M at the base to 0.5 M at the top. The addition of one-half the moment calculated at the base in accordance with Sec. 4.2 and 4.5 is based on analyses of inverted pendulums covering a wide range of practical conditions.

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

This Section is intended to cover the effects of vertical ground motion where they are most important. Factors of safety provided for gravity load design, coupled with the small likelihood that maximum live loads and earthquake loads would occur simultaneously, introduce some protection against the effects of the vertical component of ground motion. Consequently there is need for special design for vertical ground accelerations only when the effects are significant when compared with those from horizontal accelerations.

Requirements for providing protection against the possible effects of the vertical component of earthquake motions are given. In the case of standard structures, these effects are taken into account by a variation of 20 percent which is placed on the dead load (see Sec. 3.7.1). A reduction in the gravity forces due to the response to the vertical component of ground motions can be considerably more detrimental in the case of prestressed horizontal components for similar but regularly reinforced concrete components. Thus it is recommended that the 20 percent variation in dead load be replaced by a 50 percent variation. To account for the effects of vertical vibration of horizontal cantilever members, it is recommended that they be designed for a net upward force of 0.2 Q_D. The structural members most vulnerable to vertical earthquake forces are prestressed and cantilevered beams, girders, and slabs.

The specific procedures are based in part on the premise that the vertical accelerations which would develop in a building are very close to those corresponding to a structure which is perfectly rigid in the vertical direction. This is a reasonable basis provided the horizontal structural members can develop moderate ductility factors. Design requirements presented elsewhere in these provisions would usually ensure such ductility capacity for downward inertia forces. To achieve it for upward inertia forces, connections in precast concrete structures and reinforcement in concrete members should be capable of resisting at least some reversal of vertical forces. This is not automatically fulfilled by simply supported or cantilevered beams, girders, and slabs, nor by many prestressed concrete members.

Sec. 3.8 DEFLECTION AND DRIFT LIMITS

This Section provides procedures for the limitation of story drift. The term "drift" has two connotations:

a. "Story drift" is the maximum lateral displacement within a story, i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads.

b. The lateral displacement or deflection due to design forces is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering the seismic separation requirements.

There are many reasons for controlling drift; one of these is the control of member inelastic strain. Although use of drift limitations is an imprecise and highly variable way of controlling strain, this is balanced by current state of knowledge of what the strain limitations should be.

Considerations of stability dictate that flexibility be controlled. The stability of members under elastic and inelastic deformation caused by earthquakes is a direct function of both axial loading and bending of members. A stability problem is resolved by limiting the drift on the vertical load carrying elements and the resulting secondary moment from this axial load and deflection (frequently called the P-delta effect). Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from the P-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

Buildings subjected to earthquakes need drift control to restrict damage to partitions, and shaft and stair enclosures; glass and other fragile nonstructural elements; and, most importantly, to minimize differential movement demands on the seismic safety elements. As general damage control for economic reasons is not a goal of this document and since the state of the art is not well developed in this area, the drift limits have been established without regard to considerations such as present worth of future repairs versus additional structural costs to limit drift. These are separate matters for building owners and designers to examine. To the extent that life might be excessively threatened, general nonstructural damage to nonstructural and seismic safety elements is a drift limit consideration.

The design story drift limits of Table 3-C are consensus judgments taking into account all the goals of drift control as outlined above. In terms of the objectives regarding life safety and damage control, it is felt that they will yield a substantial, though not absolute, measure of safety for well detailed and constructed brittle elements and tolerable limits wherein the seismic safety elements can successfully perform, provided they are designed and constructed in accordance with these provisions.

To provide a higher performance standard, the drift limit for the essential facilities of Seismic Hazard Exposure Group III is more stringent than the limit for Groups I and II.

The drift limit for the structures of ordinary importance in Seismic Hazard Exposure Group I can be relaxed somewhat provided the criteria of the footnote to Table 3-C are met. The type of building envisioned would be similar to a prefabricated steel structure with metal skin. When the one-third increase is used, it is recommended that special provisions be provided for the seismic safety elements to accommodate the drift.

It should be emphasized that the drift limits, Δ_a, of Table 3-C are story drifts and therefore applicable to each story, i.e., they shall not be exceeded in any story even though the drift in other stories may be well below the limit. The limit, Δ_a is to be compared to the design story drift as determined by Sec. 4.6.1.

Stress or strength limitations imposed by design level forces may occasionally provide adequate drift control. However, it is expected that the design of moment resisting frames, especially steel building frames, and the design of tall, narrow shear walls or braced frame buildings will be governed at least in part by drift considerations. In areas having a large seismic coefficient, A_v, it is expected that seismic drift considerations will predominate for buildings of medium height. In areas having a low seismic coefficient and for very tall buildings in areas with large coefficients, wind considerations may generally control, at least in the lower stories.

Due to probable first mode drift contributions and C_s being generally conservative at higher values of T or T_a, the ELF procedure of Chapter 4 may be too conservative for drift design of very tall moment-frame buildings. It is suggested for these buildings, where the first mode would be responding in the displacement region of a response spectra (where displacements would be essentially independent of stiffness), that the Modal Analysis Procedures of Chapter 5 be used for design even when not required by Sec. 3.5.

Building separations and seismic joints are separations between two adjoining buildings or parts of the same building, with or without frangible closures, for the purpose of permitting the adjoining buildings or parts to respond independently to earthquake ground motion. Unless all portions of the structure have been designed and constructed to act as a unit, they must be separated by seismic joints. It is recommended that unless irregular structures can be reliably expected to act as a unit, seismic joints be utilized to separate the building into units whose independent response to earthquake ground motion can be predicted.

Although the provisions do not give precise formulations for the separations, it is required that the distance be "sufficient to avoid damaging contact under total deflection" in order to avoid interference and possible destructive hammering between buildings. It is recommended that the distance be equal to the total of the lateral deflections of the two units assumed deflecting towards each other (this involves increasing separations with height). If the effects of hammering can be shown not to be detrimental, these distances can be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, it is suggested that older code requirements for structural separations of at least one inch (1") plus one-half inch (1/2") for each ten feet (10') of height above twenty feet (20') be followed.

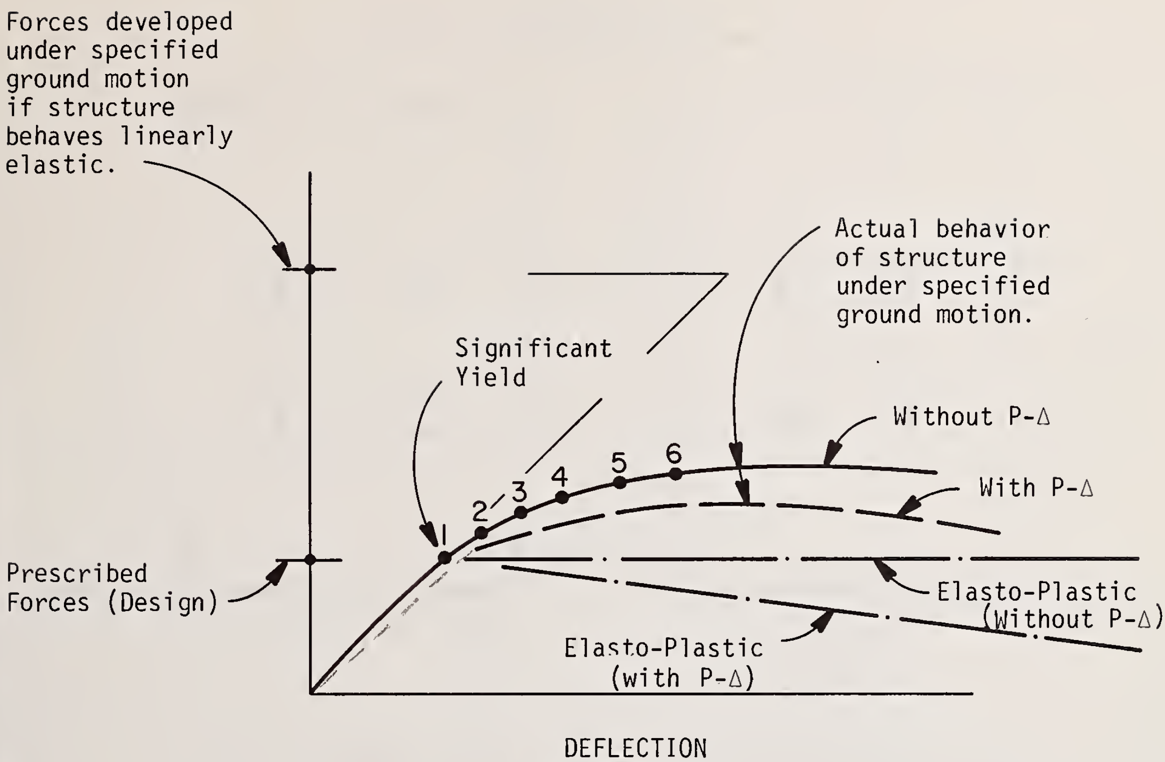
$(1'' + .5'')(\frac{h}{10'})$ or $1'' + .5''(\frac{h}{10}) = 3.5''$

So $50' = h \Rightarrow 7.5''$

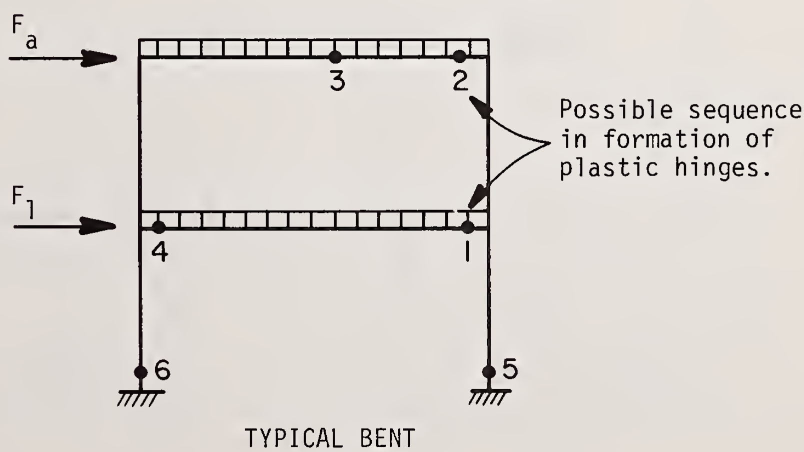
$.015(50') = 9'' \times 2 = 18''$

$1''$ @ $20'$

@ $50' \Rightarrow 1'' + .5''(\frac{50-20}{10}) = 2.5''$

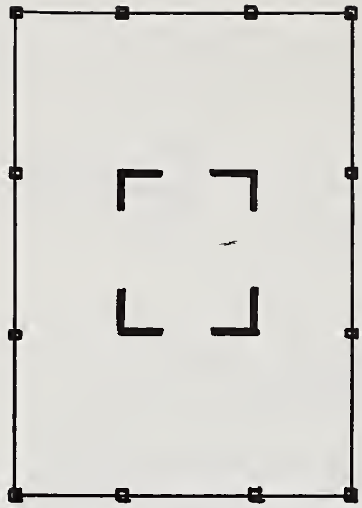


(a)



(b)

FIGURE C3-1



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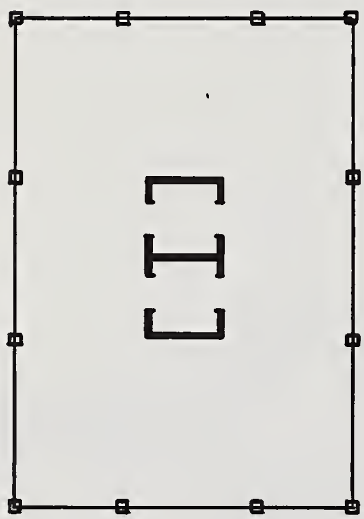
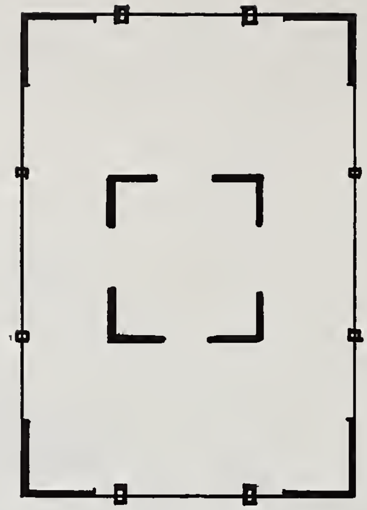


(b)

FIGURE C3-2



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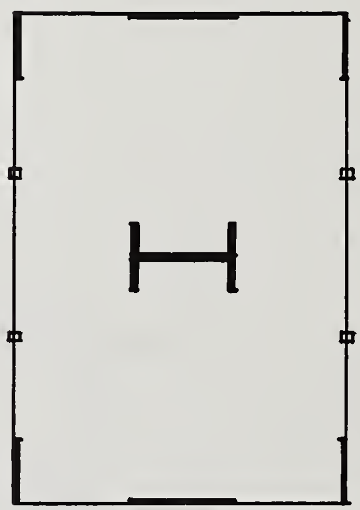
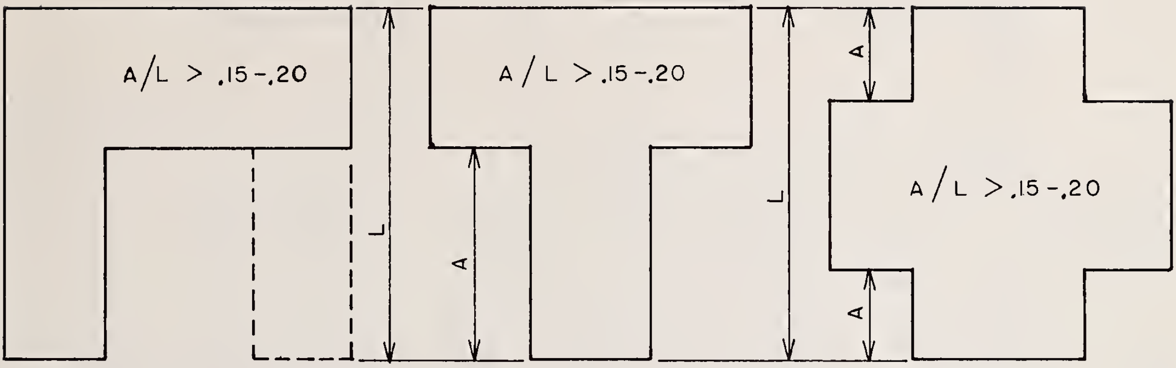


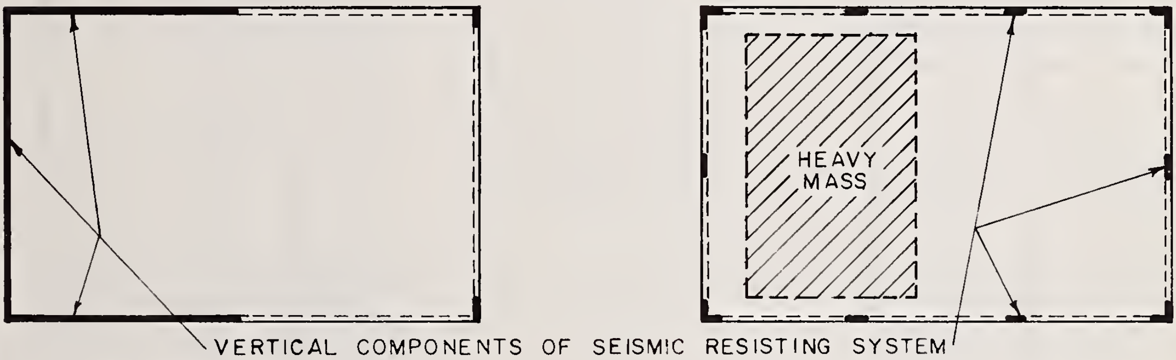
(b)

FIGURE C3-3

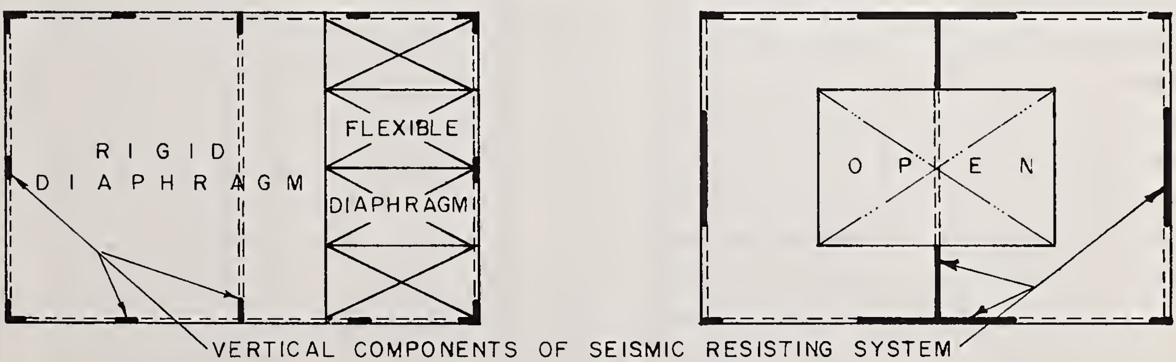
PLAN IRREGULARITIES



GEOMETRY



MASS-RESISTANCE ECCENTRICITY



DISCONTINUITY IN DIAPHRAGM STIFFNESS

FIGURE C3-4

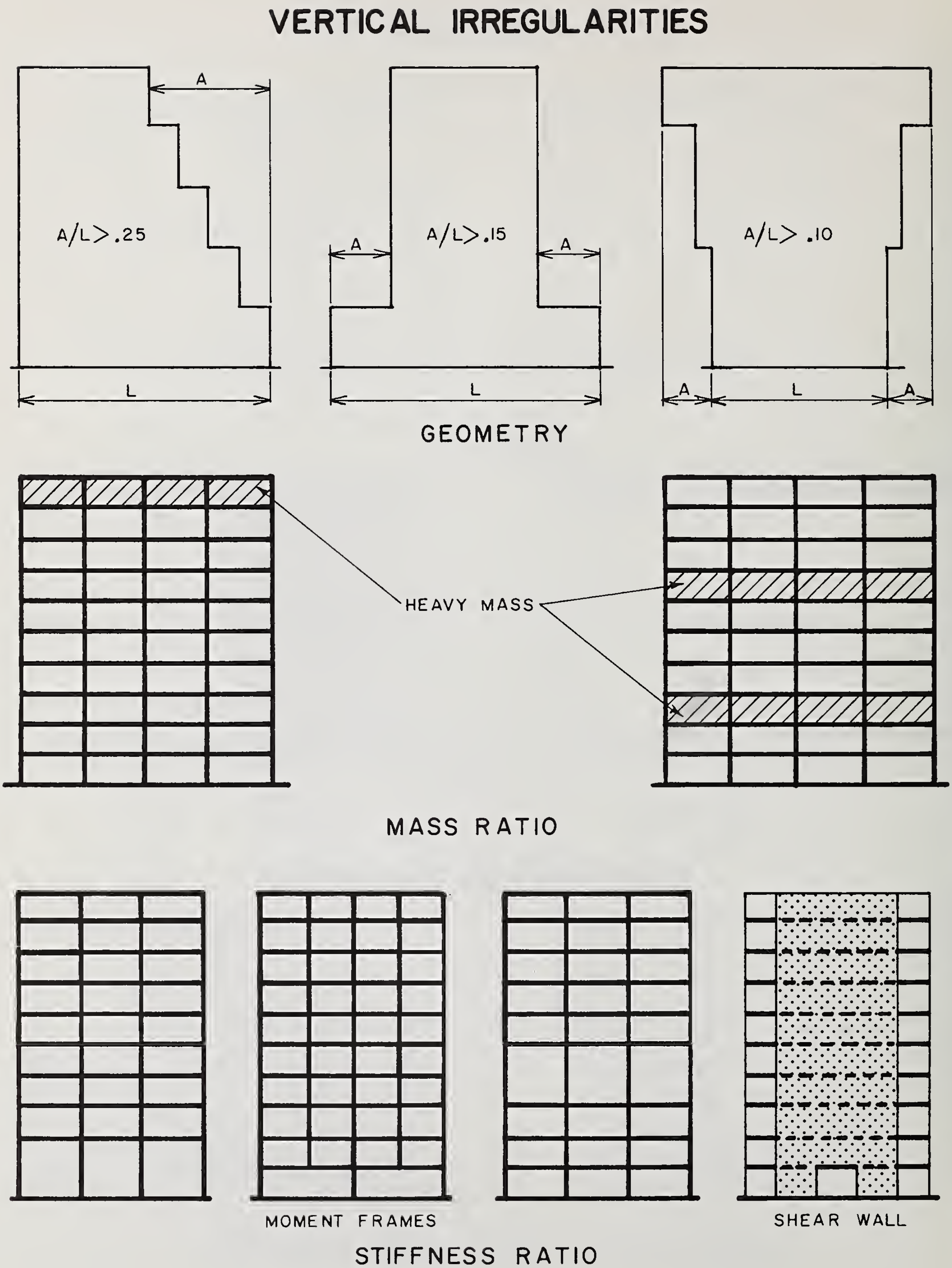
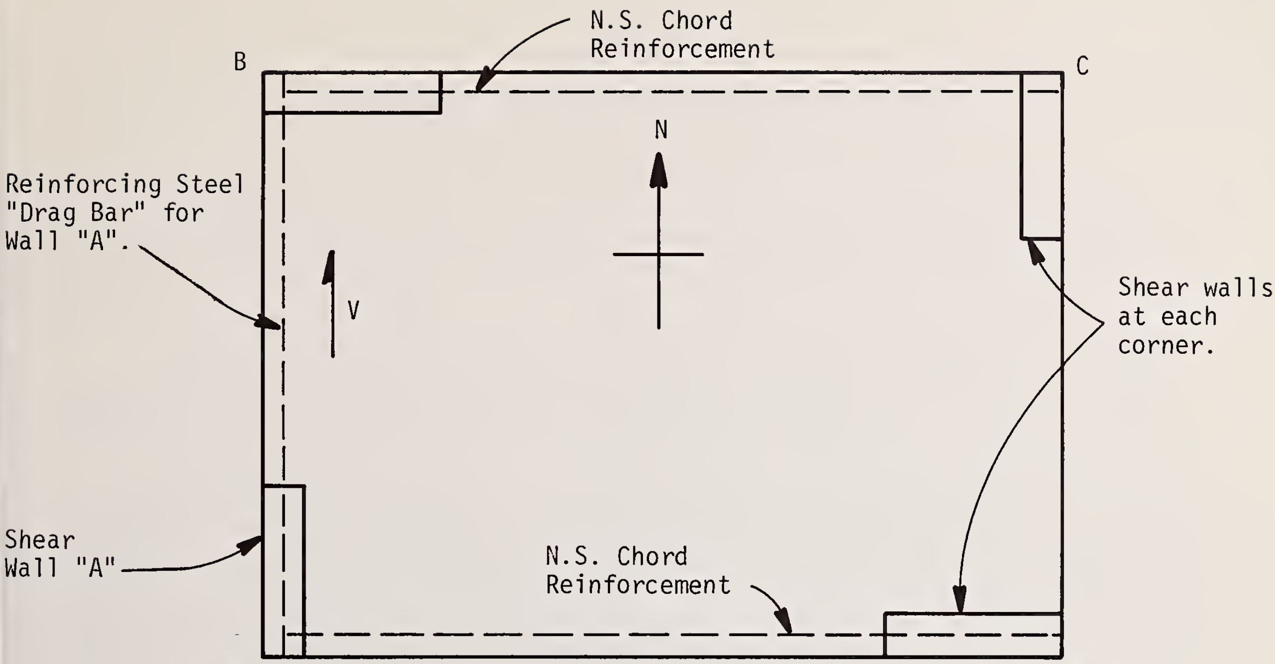

VERTICAL IRREGULARITIES
A
A/L>.25
L
A
A/L>.15
A
L
A/L> .10
A
L
A
GEOMETRY
HEAVY MASS
MASS RATIO
MOMENT FRAMES
SHEAR WALL
STIFFNESS RATIO

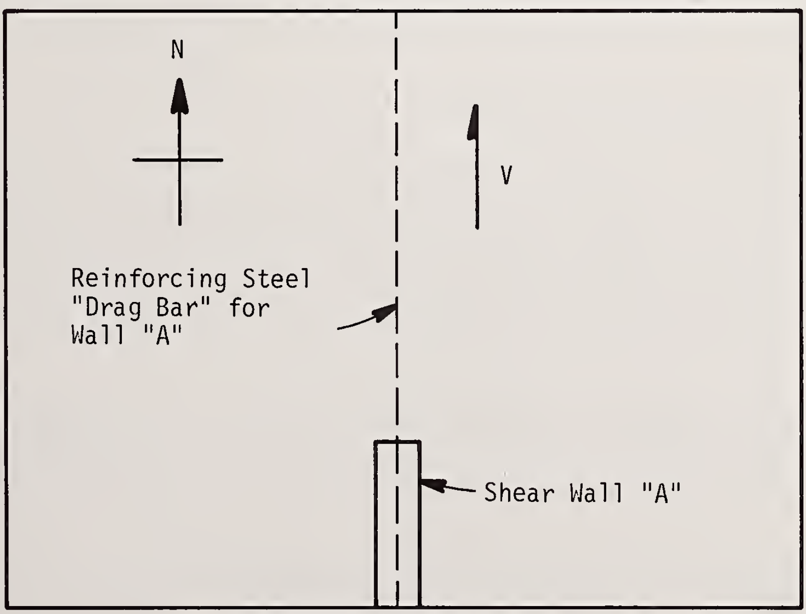
FIGURE C3-5



PLAN OF BUILDING

$\frac{WL^2}{8}$ $\frac{PL}{8}$

FIGURE C3-6



PLAN OF BUILDING

FIGURE C3-7

COMMENTARY

CHAPTER 4: EQUIVALENT LATERAL FORCE PROCEDURE

Sec. 4.1 GENERAL

This chapter covers the equivalent lateral force seismic analysis procedure for buildings.

Sec. 4.2 SEISMIC BASE SHEAR

The heart of equivalent lateral force (ELF) procedure is Formula 4-1 for base shear, which gives the total seismic design force V in terms of two factors, a seismic coefficient, C_s, and the total gravity load of the building, W.

The gravity load W is the total weight of the building and that part of the service load that one might reasonably expect to be attached to the building at the time of an earthquake. This includes partitions, permanent or movable, plus permanent equipment such as mechanical and electrical equipment, piping, ceilings, etc. The normal human live load is taken to be negligibly small in its contribution to the seismic lateral forces. Buildings designed for storage or warehouse usage shall have at least 25 percent of the design floor live load included in the weight, W. Snow loads up to 30 psf are not considered (Sec. 2.1). Freshly fallen snow would have little effect on the lateral force in an earthquake; however, ice loading would be more or less firmly attached to the roof of the building and would contribute significantly to the inertia force. For this reason, the effective snow load is taken as the full snow load for those regions where the snow load exceeds 30 psf with the proviso that the local Regulatory Agency may allow the snow load to be reduced up to 75 percent. The question of how much snow load should be included in W is really a question of how much ice build up could be expected at the building site, and this is a question best left to the discretion of the local Regulatory Agency.

The seismic coefficient formula and the various factors contained therein were arrived at on the following bases.

Elastic Acceleration Response Spectra: See Commentary for Chapter 1.4.1.

Elastic Design Spectra: It is apparent from the foregoing paragraphs that the elastic acceleration response spectra for earthquake motions has a descending branch for longer values of T, the period of vibration of the system, and it varies roughly as 1/T. However, because of a number of reasons associated with structural behavior of long-period buildings, it was decided that ordinates of design spectra should not decrease as rapidly with T; hence, the period T appears to the two-third power in the denominator of Formula 4-2. The reasons for designing long-period buildings more conservatively include the following:

1. The fundamental period of a building increases with number of stories. Hence, the longer the T, the larger the likely number of stories and therefore the number of degrees of freedom; hence, the more likely that high ductility requirements can be concentrated in a few stories of the building, at least for some earthquakes.

2. The number of potential modes of failure increases, generally with T. If design spectra were proportional to response spectra for single-degree-of-freedom systems, the probability of failure would increase with T.

3. Instability of a building is more of a problem with increasing T.

Estimated Period: T in the denominator of Formula 4-2 is intended to be an estimate of the fundamental period of vibration of the building. Methods of mechanics cannot be employed to calculate the vibration period before a design of the building, at least a preliminary one, is available. Simple formulas which involve only a general description of the building type (e.g., steel moment frame, concrete moment frame, shear wall system, braced frame, etc.), and overall dimensions (such as height and plan length) are therefore necessary to estimate the vibration period in order to calculate an initial base shear and proceed with a preliminary design. For preliminary member sizing, it is advisable that this base shear and the corresponding value of T be conservative. Thus, the value of T should be smaller than the true period of the building. Formulas 4-4 and 4-5 are therefore intended to provide conservative estimates of the fundamental period of vibration.

Taking the seismic base shear coefficient to vary as $1/T^{2/3}$, and assuming that the lateral forces are distributed linearly over the height and that the deflections are controlled by drift limitations, a simple analysis of the vibration period by Rayleigh's method (4,5,6,7)* leads to the conclusion that the vibration period of moment-resisting structures varies roughly as $h_n^{3/4}$, where h_n = total height of the building as defined elsewhere. Formula 4-4 is therefore appropriate and the values of the coefficient C_T have been established to produce values for T_a generally lower than the true fundamental vibration period of moment frame buildings. This is apparent in Figures C4-1 and C4-2, wherein Formula 4-4 is compared with fundamental vibration periods as computed from accelerograph records from upper stories of several buildings during the 1971 San Fernando earthquake.

Formula 4-5 is identical to an existing formula in the SEAOC recommendations (1). It is apparent from Figure C4-3 that this would generally underestimate the fundamental vibration period of reinforced-concrete shear-wall buildings. Formula 4-5 is to be used for all buildings other than those included in Figures C4-1 to C4-3 because there is insufficient data on measured periods of such building types and materials to permit development of special formulas. It is expected to provide underestimates of periods of vibration for other building types.

As an exception to Formulas 4-4 and 4-5, these design provisions allow the calculated fundamental period of vibration T of the seismic resisting system to be used in calculating the base shear. However, the period, T, used may not exceed 1.2 T_a as determined from Formula 4-4 or 4-5 as appropriate.

For exceptionally stiff or light buildings, the calculated T for the seismic resisting system may be significantly shorter than T_a calculated by Formula 4-4 or 4-5. For such buildings it is recommended that the period value T be used in lieu of T_a for calculating the base shear coefficient C_s.

The fundamental period of vibration of the seismic resisting system is to be calculated according to established methods of mechanics (4,5,6,7). Computer programs are available for such calculations. One method of calculating the period, probably as convenient as any, is the use of a formula based on Rayleigh's method (4,5,6,7).

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \qquad \text{(C4-1)}$$

*See References at end of Chapter 5 Commentary.

in which F_i is the seismic lateral force at level i, w_i is the gravity load assigned to level i, δ_i is the static lateral displacement at level i due to the forces F_i computed on a linear elastic basis, and g is the acceleration of gravity.

The calculated period increases with an increase in flexibility of the structure, for the δ term in the Rayleigh formula appears to the second power in the numerator but to only the first power in the denominator. Thus if, in calculating the deflections δ, one ignores the contribution of nonstructural elements to the stiffness of the structure, the deflections are exaggerated and the calculated period is lengthened, leading to a decrease in the coefficient C_s and therefore a decrease in the design force. Nonstructural elements do not know that they are nonstructural. They participate in the behavior of the structure, even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side. The limitation of 1.2 T_a is imposed as a safeguard. If the ratio were this maximum of 1.2, the effects on design lateral forces would be a reduction of less than 10 percent.

Response Modification Factor: The factor R in the denominator of Formula 4-2 is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Thus for a lightly damped building structure of brittle material which would be unable to tolerate any appreciable deformation beyond the elastic range, the factor R would be close to 1; i.e., no reduction would be allowed. At the other extreme, a heavily damped building structure with a very ductile structural system would be able to withstand deformations considerably in excess of initial yield and would therefore justify the assignment of a larger response reduction factor R. Table 3-B in the provisions stipulates R coefficients for different types of building systems using several different structural materials. The coefficient R ranges in value from a minimum of one and one-fourth for an unreinforced masonry bearing wall system to a maximum of eight for a Special Moment Frame system. The basis for the R-factor values specified in Table 3-B is presented in Commentary for Chapter 3.

Formulas 4-3 and 4-3a provide a cut-off for lower period buildings. A discussion of these two formulas is given in the Commentary for Sec. 1.4.1.

During the discussions leading to the establishment of Formulas 4-1 for determining the design base shear of a building, the use of a factor (such as an occupancy factor) related to the Seismic Hazard Exposure Group (SHE) was considered. After lengthy consideration the committee decided that arbitrarily increasing the seismic base shear is generally ineffective in improving building safety. Good connections and construction details, quality assurance procedures, and limitations on building deformation or drift will significantly improve the capability for maintenance of function and safety in critical facilities and those with high density of occupancy. Accordingly the committee, after comparing the design effects resulting from the ATC-specified provisions with previous design codes, decided that the specified force levels provide an adequate force function for design of all buildings. However, to improve the capability for meeting the more restrictive requirements for SHE Group II buildings, building design categories were specified and appropriate special detailing requirements added. The reduction in the damage potential of critical facilities (SHE Group III) was handled by using more conservative drift controls (Sec. 3.8) and by providing special design and detailing requirements (Sec. 3.6), and materials limitations (Chapters 9 through 12).

Sec. 4.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The distribution of lateral forces over the height of a building is generally quite complex because these forces are the result of superposition of a number of natural modes of vibration. The relative contributions of these vibration modes to the total forces depends on a number of factors, including shape of the earthquake response spectrum, natural periods of vibration of the building, and shapes of vibration modes which in turn depend on the mass and stiffness distribution over the height of the building. Sec. 4.3 provides a reasonable and simple method for determining the lateral force distribution in buildings with regular variation of mass and stiffness over the height, see Sec. 3.4. The basis of this method is discussed in the following paragraphs. In buildings having only minor irregularity of mass or stiffness over the height, the accuracy of the lateral force distribution as given by Formula 4-6a is much improved by the procedure described under Sec. 3.5 of this Commentary.

The lateral force at each floor, x, due to response in the first (fundamental) natural mode of vibration is

$$f_{x1} = V_1 \frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_i \phi_{i1}}$$

where V_1 is the contribution of this mode to the base shear, w_i is the weight lumped at the ith floor level, and ϕ_i is the amplitude of the first mode at the ith floor level. This is the same as Formulas 5-4 and 5-4a in Chapter 5 of the provisions but specialized for the first mode. If V_1 is replaced by the total base shear, V, the above formulas will become identical to Formulas 4-6 and 4-6a with k = 1 if the first mode shape is a straight line, and with k = 2 if the first mode shape is a parabola with its vertex at the base.

It is well known that the influence of modes of vibration higher than the fundamental mode is small in the earthquake response of short-period buildings and that in regular buildings the fundamental vibration mode departs little from a straight line. This along with the foregoing paragraph provides a basis for Formula 4-6a; with k = 1 for buildings having a fundamental vibration period of 0.5 second or less.

It has been demonstrated that although the earthquake response of long-period buildings is primarily due to the fundamental natural mode of vibration, the influence of higher modes of vibration can be significant, and in regular buildings the fundamental vibration mode lies approximately between a straight line and a parabola with the vertex at the base. In light of this and the foregoing paragraph, Formula 4-6a with k = 2 is appropriate for buildings having a fundamental period of vibration of 2.5 seconds or longer. Linear variation of k between 1 at 0.5 second period and 2 at 2.5 seconds provides the simplest possible transition between the two extreme values.

Sec. 4.4 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

The torsional moment to be considered in the design of elements in a story consists of two parts:

1. M_t, the moment due to eccentricity between centers of mass and resistance for that story, shall be computed as the story shear times the eccentricity perpendicular to the direction of applied earthquake forces.

2. M_{ta}, commonly referred to as "accidental torsion," shall be computed as the story shear times the "accidental eccentricity" equal to 5 percent of the dimension of the building, in the story under consideration perpendicular to the direction of the applied earthquake forces.

Computation of M_{ta} in this manner is equivalent to the procedure in Sec. 4.3, wherein it is implied that the "dimension of the building" is the dimension in the story where the torsional moment is being computed and that all the masses above the story should be assumed to be displaced in the same direction at one time, say first all of them to the left and then to the right.

Dynamic analyses assuming linear behavior indicate that the torsional moment due to eccentricity between centers of mass and resistance may significantly exceed M (4). However, such dynamic magnification is not included in these design provisions, partly because its significance is not well understood for buildings designed to deform well beyond the range of linear behavior.

The torsional moment M_t calculated in accordance with this provision would be zero in those stories where centers of mass and resistance coincide. However, during vibration of the building, torsional moments would be induced in such stories due to eccentricities between centers of mass and resistance in other stories. To account for such effects, it is recommended that the torsional moment in any story be not smaller than the following two values: the story shear times one-half of the maximum of the computed eccentricities in all stories below the one being analyzed, and one-half of the maximum of the computed torsional moments for all stories above (4).

Accidental torsion is intended to cover the effects of several factors which have not been explicitly considered in the design provisions. These factors include the rotational component of ground motion about a vertical axis; unforeseeable differences between computed and actual values of stiffness, yield strengths and dead-load masses; and unforeseeable unfavorable distributions of live-load masses.

There are indications that the 5-percent accidental eccentricity may be too small in some buildings for they may develop torsional dynamic instability. Some examples are the upper stories of tall buildings having little or no nominal eccentricities, those structures where the calculations of relative stiffnesses of various elements are particularly uncertain, such as those that depend largely on masonry walls for lateral-force resistance or those that depend on vertical elements made of different materials, and nominally symmetrical structures which behave essentially like elastic nonlinear systems, such as some prestressed concrete frames. In such cases, it will be appropriate to increase the accidental eccentricity from 5 to perhaps 10 percent of the appropriate building dimension as discussed previously.

The way in which the story shears and the effects of torsional moments are distributed to the vertical elements of the seismic resisting system depends on the stiffness of the diaphragms relative to vertical elements of the seismic resisting systems.

Where the diaphragm stiffness in its own plane is sufficiently high relative to the stiffness of the vertical components of the seismic resisting system, the diaphragm may be assumed to be infinitely rigid for purposes of this section. Then, in accordance with compatibility and equilibrium requirements, the shear in any story shall be distributed among the vertical components in proportion to their contributions to the lateral stiffness of the story, while the story torsional moment produces additional shears in these components which are proportional to their contributions to the torsional stiffness of the story about its center or resistance. This contribution of any component is the product of its lateral stiffness and the square of its distance to the center of resistance of the story. Alternatively, the story shears and torsional moments may be distributed on the basis of a three-dimensional analysis of the structure, consistent with the assumption of linear behavior.

Where the diaphragm in its own plane is very flexible relative to the vertical components, each vertical component acts almost independently of the rest; accidental torsion is insignificant and can therefore be ignored. The story shear should be distributed to the vertical components considering these to be rigid supports. Analysis of the diaphragm acting as a continuous horizontal beam or truss on rigid supports leads to the distribution of shears. Because the properties of the beam or truss may not be accurately computed, it is recommended that the shears in vertical elements not be taken less than those based on tributary areas.

There are some common situations where it is obvious whether the diaphragm can be assumed as rigid or very flexible in its own plane for purposes of distributing story shear and considering torsional moments. For example, a solid monolithic, reinforced concrete slab, square or nearly square in plan, in a building with slender, moment resisting frames, may be regarded as rigid; a timber floor on squatty masonry bearing walls may be regarded as very flexible. In intermediate situations it is recommended that the design forces be based on an analysis which explicitly considers diaphragm deformations and satisfies equilibrium and compatibility requirements, or they should be the envelope of the two sets of forces resulting from both extreme assumptions regarding the diaphragm: infinitely stiff or very flexible.

Where the horizontal diaphragm is not continuous, the story shear can be distributed to the vertical components based on their tributary areas and torsional moments (both M_t and M_{ta}) can be ignored.

Sec. 4.5 OVERTURNING

This section contains the requirement that the building be designed to resist overturning moments statically consistent with the design story shears, except for reduction factor κ in Formula 4-8. There are several reasons for reducing the statically computed overturning moments:

1. The distribution of design story shears over height computed from the lateral forces of Sec. 4.2 is intended to provide an envelope; shears in all stories do not attain their maximum simultaneously. Thus the overturning moments computed statically from the envelope of story shears will be overestimated.

2. It is intended that the design shear envelope, which is based on the simple distribution of forces specified in Sec. 4.3, be conservative. If the shear in a specific story is close to the exact value, the shears in almost all other stories are almost necessarily overestimated. Hence, the overturning moments statically consistent with the design story shears will be overestimated.

3. Under the action of overturning moments, one edge of the foundation may lift from the ground for short durations of time. Such behavior leads to substantial reduction in the seismic forces and consequently the overturning moments.

The overturning moments computed statically from the envelope of story shears may be reduced by no more than 20 percent. This value is similar to those obtained from results of dynamic analysis taking into account the first two reasons of the foregoing paragraph (4). No reduction is permitted in the uppermost ten stories primarily because the statically computed overturning moment in these stories may err on the unsafe side (4). In any case, there is hardly any benefit in reducing the overturning moments in the stories near the top of buildings, because design of vertical elements in these stories is rarely governed by overturning moments. For the 11th to 20th stories from the top, linear variation of κ provides the simplest transition between the minimum and maximum values of 0.8 and 1.0.

In the design of the foundation, the overturning moment may be calculated at the foundation-soil interface using Formula 3-6 with $\kappa = 0.75$ for all building heights. This is appropriate because a slight uplifting of one edge of the foundation during vibration leads to reduction in the overturning moment, and because such behavior does not normally cause structural distress.

Formerly many building codes and design recommendations, including the 1968 SEAOC recommendations (2), allowed more drastic reduction in overturning moments relative to their value statically consistent with the design story shears. These reductions appeared to be excessive in light of the damage to buildings during the 1967 Caracas earthquake, where a number of column failures were primarily due to effects of overturning moment. In later versions of the SEAOC recommendations (3), no reduction was allowed. The moderate reduction permitted in Sec. 4.5, which is consistent with results of dynamic analyses (4), is more appropriate because use of the full statically determined overturning moment can not be justified in light of the reasons mentioned in the first paragraph.

Sec. 4.6 DRIFT DETERMINATION AND P-DELTA EFFECTS

This section defines the design story drift as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe}, (determined from an elastic analysis) by the deflection amplification factor, C_d, as given in Table 3-B. The elastic analysis is to be made for the seismic resisting system using the prescribed seismic design forces and considering the building to be fixed at the base. Stiffnesses other than those of the seismic resisting system should not be included as they may not be reliable at higher, inelastic strain levels.

The deflections shall be determined by combining the effects of joint rotation of members, shear deformations between floors, the axial deformations of the overall lateral resisting elements, and the shear and flexural deformations of shear walls and braced frames. The deflections are determined initially on the basis of the distribution of lateral forces stipulated in Sec. 4.3. For frame structures, the axial deformations from bending effects, although contributing to the overall building distortion, may or may not affect the story to story drift. However, they shall be considered. Centerline dimensions between the frame elements are often used for analysis, but clear span dimensions with consideration of joint panel zone deformation may also be used.

For determining compliance with the story drift limitation of Sec. 3.8, the deflections, δ_x, may be calculated as above, or the seismic resisting system and design forces corresponding to the fundamental period of the building, T, (calculated without the limit specified in Sec. 4.2.2) may be used. The same model of the seismic resisting system used in determining the deflections must be used for determining T. The waiver does not pertain to the calculation of drifts for determining P-delta effects on member forces, overturning moments, etc. If the P-delta effects as determined in Sec. 4.6.2 are significant, the design story drift shall be increased by the resulting incremental factor.

The P-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces prescribed in Sec. 4.3 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the P-delta moment to the lateral force story moment is designated as a stability coefficient θ in Formula 4-10. If the stability coefficient θ is less than 0.10 for every story, then the P-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient θ exceeds 0.10 for any story, then the P-delta effects on story drifts, shears, member forces, etc., for the whole building must be determined by a rational analysis.

An acceptable P-delta analysis, when required, is as follows:

1. Compute for each story the P-delta amplification factor, $a_d = \theta/(1-\theta)$. a_d takes into account the multiplier effect due to the initial story drift leading to another increment of drift which would lead to yet another increment, etc. Thus both the effective shear in the story and the computed eccentricity would be augmented by a factor $1 + \theta + \theta^2 + \theta^3...$, which is $1/(1-\theta)$ or $(1 + a_d)$.

2. Multiply the story shear V_x in each story by the factor $(1 + a_d)$ for that story and recompute the story shears, overturning moments, and other seismic force effects corresponding to these augmented story shears.

Any of a number of rational analyses could be used. Some published computer programs take P-delta effects into account.

The augmented story drifts thus determined are the drifts that would pertain to an elastic structure. The drifts characterizing the extreme displacement expected from the design earthquake would be magnified because of inelastic displacement. Therefore, the design story drifts are stipulated to be those computed by Formula 4-10, which incorporates the deflection amplification factor, C_d, ranging in value from 1.25 to 6.5, depending upon the ductility of the structural system and the structural materials employed.

TABLE C4-1: STEEL FRAME BUILDINGS

(LIST FOR FIGURE C4-1)

Identification Number	Name and Address
1	K B Valley Center 15910 Ventura
2	Jet Propulsion Lab Administration Building, Bldg 180
3	6464 Sunset Boulevard
4	1900 Avenue of the Stars Century City
5	1901 Avenue of the Stars Century City
6	1880 Century Park East Century City
7	1888 Century Park East Office Tower Century City
8	Mutual Benefit Life Plaza 5900 Wilshire Boulevard
9	Department of Water & Power 111 N. Hope Street
10	Union Bank Building 445 South Figueroa
11	Kajima International 250 East First Street
12	Bunker Hill Tower 800 West First Street
13	3407 West 6th Street
14	Occidental Building 1150 South Hill Street
15	Crocker Citizens Bank Building 611 West 6th Street
16	Sears Headquarters 900 South Fremont, Alhambra
17	5260 Century Boulevard

TABLE C4-2: REINFORCED CONCRETE FRAME BUILDINGS

(LIST FOR FIGURE C4-2)

Identification Number	Name and Address
1	Holiday Inn 8244 Orion Street
2	Valley Presbyterian Hospital 15107 Vanowen Boulevard
3	Bank of California 15250 Ventura Boulevard
4	Hilton Hotel 15433 Ventura Boulevard
5	Sheraton-Universal 3838 Lankershim Boulevard
6	Muir Medical Center 7080 Hollywood Boulevard
7	Holiday Inn 1760 North Orchid
8	1800 Century Park East Century City
9	Wilshire Christian Towers 616 S. Normandie Avenue
10	Wilshire Square One 3345 Wilshire Boulevard
11	533 South Fremont
12	Mohn Olympic 1625 Olympic Boulevard
13	120 Robertson
14	Holiday Inn 1640 Marengo

TABLE C4-3: REINFORCED CONCRETE SHEAR WALL BUILDINGS

(LIST FOR FIGURE C4-3)

Identification Number	Name and Address
1	Certified Life 14724 Ventura Boulevard
2	Kaiser Foundation Hospital 4867 Sunset Boulevard
3	Millikan Library Cal Tech, Pasadena
4	1888 Century Park East Century City
5	3470 Wilshire Boulevard
6	L.A. Athletic Club Parking Structure 646 South Olive
7	Parking Struccture 808 South Olive
8	USC Medical Center 2011 Zonal
9	Airport-Marina Hotel 8639 Lincoln Marina Del Ray

SAN FERNANDO EARTHQUAKE DATA

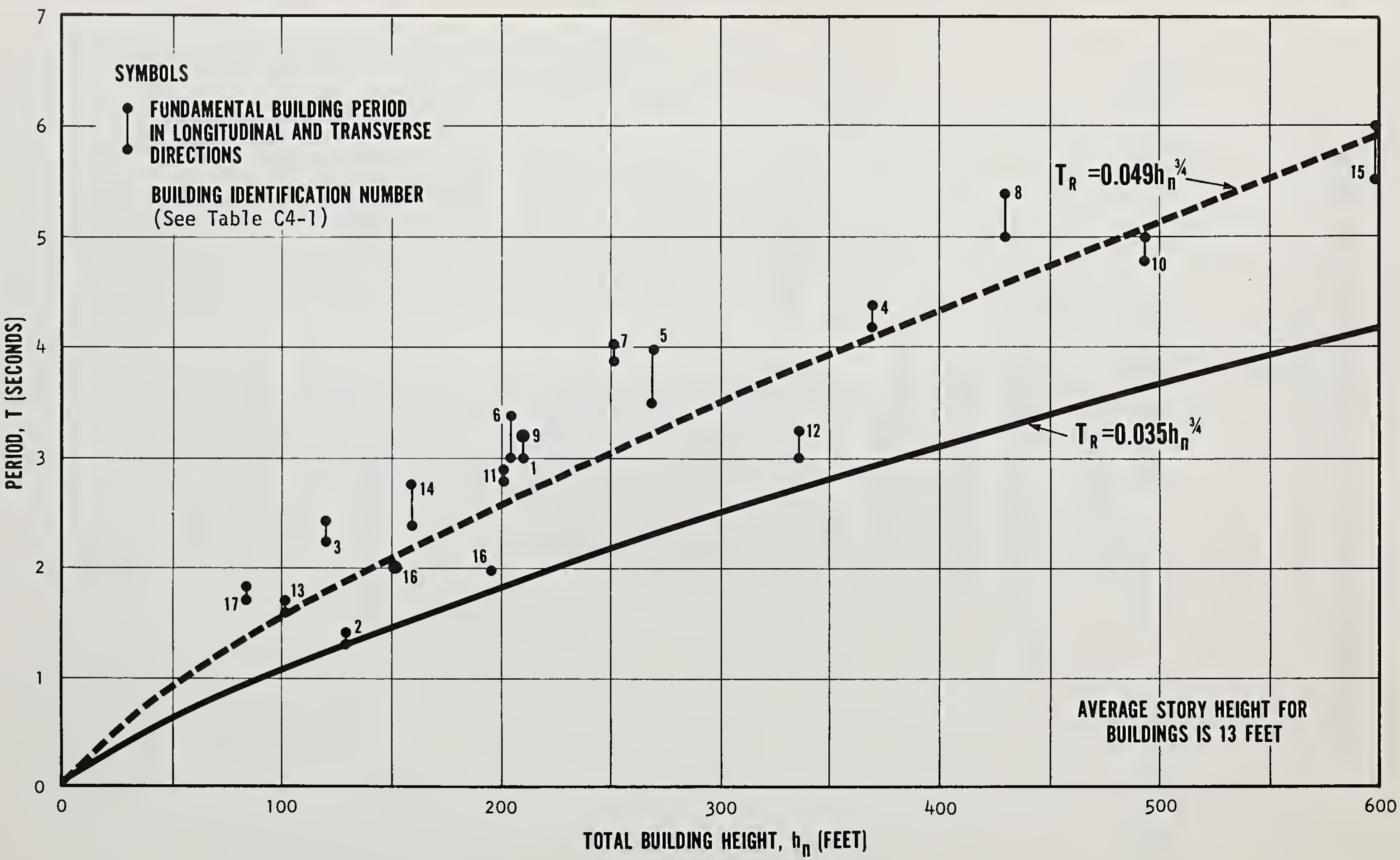


FIGURE C4-1 STEEL FRAMES

SAN FERNANDO EARTHQUAKE DATA

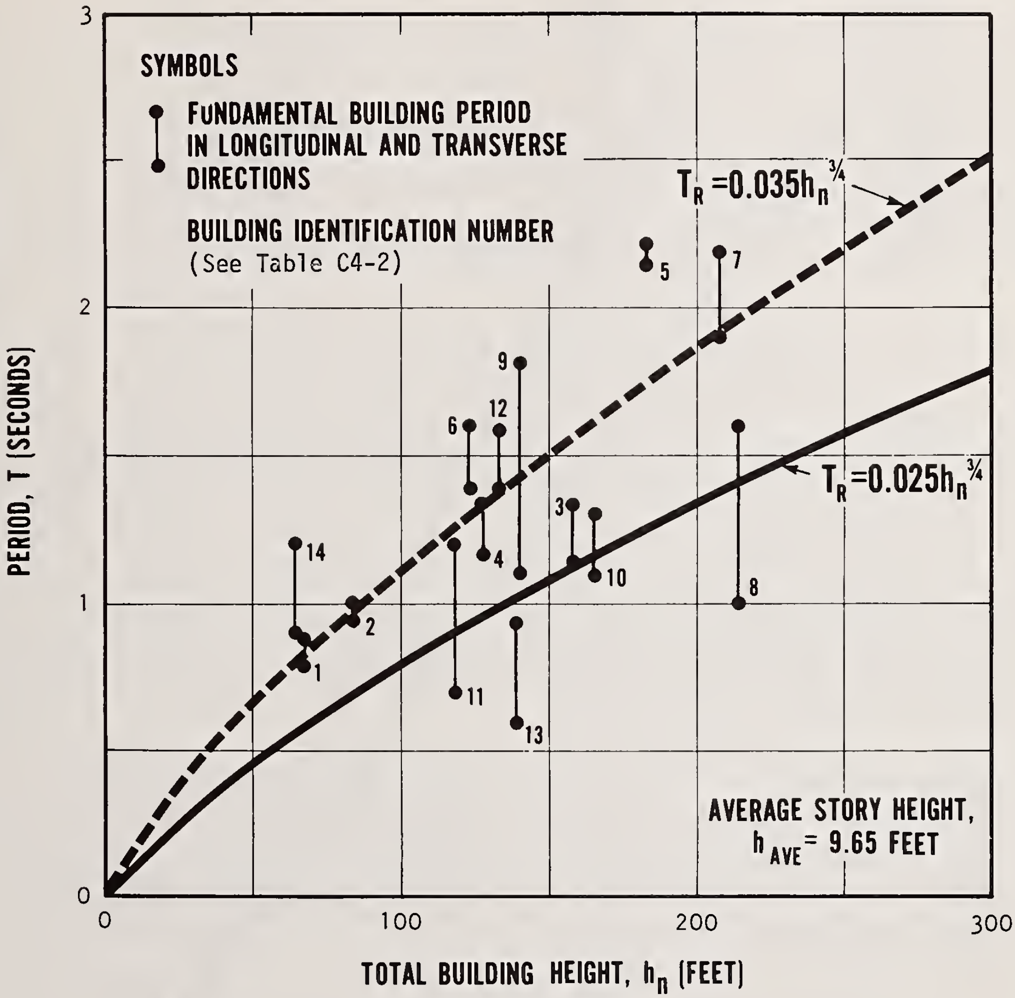


FIGURE C4-2 REINFORCED CONCRETE FRAMES

SAN FERNANDO EARTHQUAKE DATA

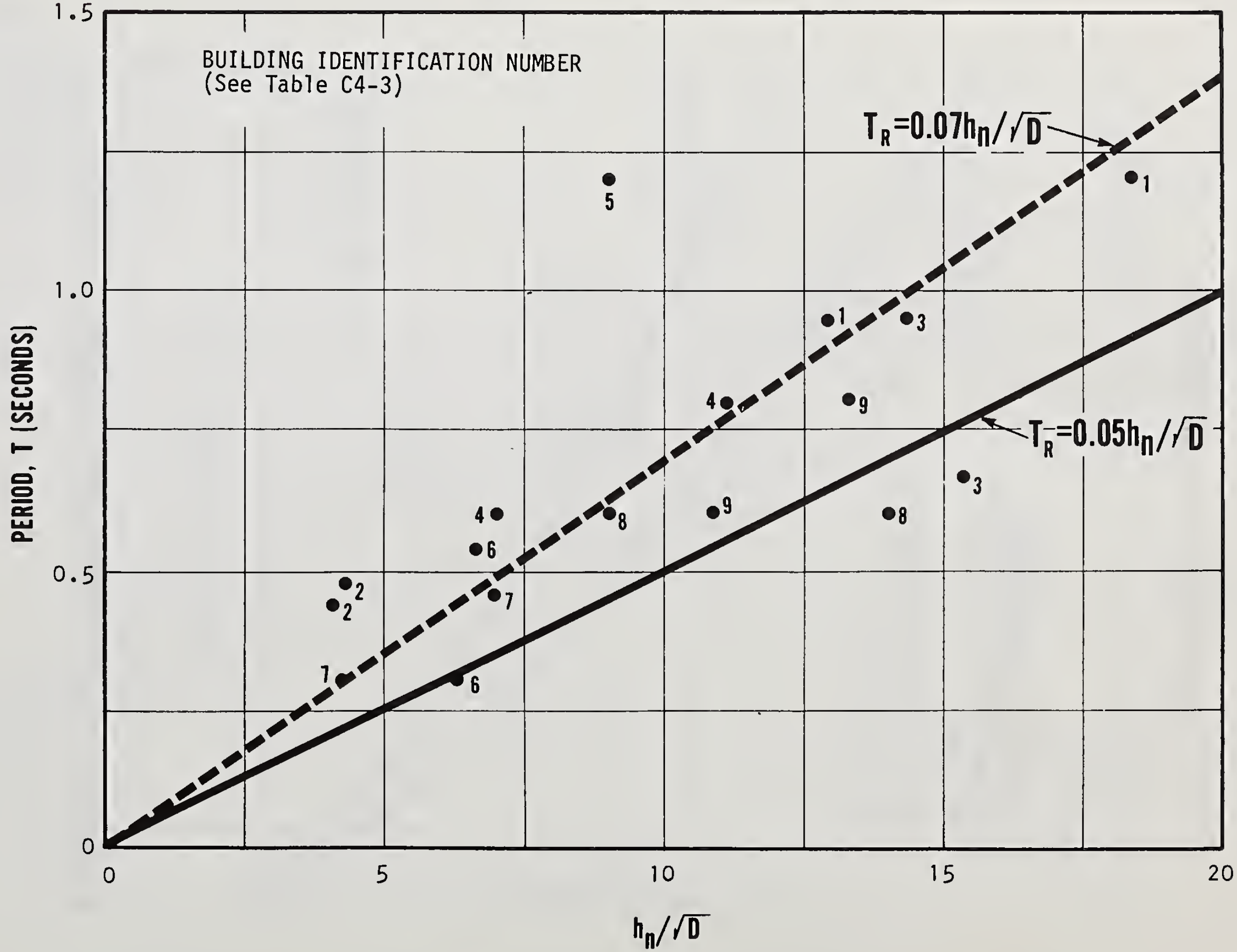


FIGURE C4-3 R/C SHEAR WALL BUILDINGS

COMMENTARY

CHAPTER 5: MODAL ANALYSIS PROCEDURE

Sec. 5.1 GENERAL and Sec. 5.2 - MODELING

Modal analysis (4-7) is generally applicable for calculating the linear response of complex, multidegree-of-freedom structures and is based on the fact that the response is the superposition of the responses of individual natural modes of vibration, each mode responding with its own particular pattern of deformation, the mode shape, with its own freqency, the modal frequency, and with its own modal damping. The response of the structure can therefore be modeled by the response of a number of single-degree-of-freedom oscillators with properties chosen to be representative of the mode and the degree to which the mode is excited by the earthquake motion. For certain types of damping, this representation is mathematically exact; and for building structures, numerous full-scale tests and analyses of earthquake response of structures have shown that the use of modal analysis, with viscously-damped single-degree-of-freedom oscillators describing the response of the structural modes, is an accurate approximation for analysis of linear response.

Modal analysis is useful in design because formulas describing seismic coefficients (e.g., Formula 4-2) can be interpreted as acceleration design spectra, and can therefore be used to specify the maximum response of each mode of a complex building. This specified maximum response can be expressed in several ways, and in the provisions it was decided that the modal forces and their distributions over the structure should be given primary emphasis to highlight the similarity to the equivalent static methods traditional in building codes (1-3). Thus the coefficient C_{sm} in Formula 5-1 and the distribution equations, Formulas 5-4 and 5-4a, are the counterparts of Formulas 4-1, 4-6, and 4-6a. This correspondence helps clarify the fact that the simplified modal analysis contained in Chapter 5 is simply an attempt to specify the equivalent lateral forces on a building in a way that directly reflects the individual dynamic characteristics of the building. Once the story shears and other response variables for each of the important modes are determined and combined to produce design values, the design values are used in basically the same manner as the equivalent lateral forces given in Chapter 4.

The modal analysis procedure specified in Chapter 5 is simplified from the general case by restricting consideration to lateral motion in a plane. Only one degree of freedom is required per floor for this type of motion, as noted in Sec. 5.2. The effects of the horizontal component of ground motion perpendicular to the direction under consideration, the vertical component of ground motion, and the torsional motions of the building are all considered in the same simple manner as in the equivalent lateral force procedure.

Sec. 5.3 MODES

The purpose of this Section is to define the number of modes to be used in the analysis. For many structures, including low-rise buildings and structures of moderate height, three modes of vibration in each direction are nearly always sufficient to determine design values of the earthquake response of the building. For buildings of only one or two stories, a number of modes equal to the number of stories suffices for purposes of design, hence the last phrase. For high-rise structures, however, more than three modes may be required to adequately determine the forces for design. In this case, all modes having natural periods larger than 0.40 seconds are to be used. For very tall or very flexible structures, it may be necessary to consider six or more modes in each direction.

The requirements of this Section are intended to specify the minimum number of modes to be conisdered and there may be instances in which the designer may wish to include additional modes in the analysis in order to obtain a more reliable indication of the possible earthquake response of the structure.

Sec. 5.4 PERIODS

Natural periods of vibration are required for each of the modes used in the subsequent calculations. These are needed to determine the modal coefficients C_{sm} from Formula 5-3. Because the periods of the modes contemplated in the provisions are those associated with moderately large, but still essentially linear, response of the building, the period calculations should include only those elements which are effective at these amplitudes. Such periods may be longer than those obtained from a small-amplitude test of the building when completed, or the response to small earthquake motions, because of the stiffening effects of nonstructural and architectural components of the building at small amplitudes. During response to strong ground shaking, however, the measured responses of buildings have shown that the periods lengthen, indicating the loss of the stiffness contributed by those components.

There exists a wide variety of methods for calculation of natural periods and associated mode shapes, and the writers of the provisions elected not to specify the particular method to be used in design. It was judged essential, however, that the method used be one based on generally accepted principles of mechanics, such as are given, for example, in well-known textbooks on structural dynamics and vibrations (4,5,6,7). Although it is expected that computer programs, whose accuracy and reliability are documented and widely-recognized, will be used to calculate the required natural periods and associated mode shapes in many cases, the use of such programs is not required.

Sec. 5.5 MODAL BASE SHEAR

A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the building vibrating in each of its important modes. As the building vibrates back and forth in a particular mode at the associated period, it experiences maximum values of base shear, interstory drifts, floor displacements, base (overturning) moments, etc. In this Section the base shear in the mth mode is specified as the product of the modal seismic coefficient C_{sm} and the effective weight $\overline{W}_m$ for the mode. The coefficient C_{sm} is determined for each mode from Formula 5-3 using the associated period of the mode, T_m, in addition to the factors A_v, S, and R which are discussed elsewhere in this Commentary. An exception to this procedure occurs for higher modes of those buildings which have periods shorter than 0.3 second and which are founded on Type S_3 soils. For such modes, Formula 5-3a is used. Formula 5-3a gives values ranging from 0.8 A_a/R for very short periods to 2.0 A_a/R for $T_m = 0.3$. Comparing these values to the limiting values of C_s of 2.0 A_a/R for Type S_3 soils as specified following Formula 5-3, it is seen that the use of Formula 5-3a, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Formula 5-3 and its limiting values. This spectrum shape so defined is a conservative approximation to average spectra which are known to first ascend, then level off, and then decay as period increases. Formula 5-3 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For Type S_1 or S_2 soils, the ascending portion of the spectra is completed by the time the periods reaches a small value near 0.1 or 0.2 seconds. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Formula 5-3a is then a replacement for the spectral shape for Type S_3 soils and short periods, which is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Formula 5-3 for modal analysis in the case of Type S_3 soils.

The effective modal gravity load given in Formula 5-2 can be interpreted as specifying the portion of the weight of the building that participates in the vibration of each mode. It is noted that Formula 5-2 gives values of $\overline{W}_m$ that are independent of how the modes are normalized.

The final equation of this Section, Formula 5-3b, is to be used if a modal period exceeds 4 seconds. It can be seen that Formulas 5-3b and 5-3 coincide at $T_m = 4$ seconds, so that the effect of using Formula 5-3b is to provide a more rapid decrease in C_{sm} as a function of T_m than implied by Formula 5-3. This modification is introduced in consideration of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods the average velocity spectrum of strong earthquake motions from large (magnitude 6.5 and larger) earthquakes is approximately horizontal, which implies that C_{sm} should decrease as $1/T_m$. Formula 5-3 decreases as $1/T_m^{2/3}$ for reasons discussed in Sec. 4.1 of this Commentary, and this slower rate of decrease, if extended to very long periods, would result in an unbalanced degree of conservatism in the modal force for very tall buildings. In addition, for very long periods, the average displacement spectrum of strong earthquake motions becomes horizontal which implies that C_{sm}, which is a form of acceleration spectrum, should decay as $1/T_m^2$. The period at which the displacement response spectrum becomes horizontal depends on the size of the earthquake, being larger for great earthquakes, and a representative period of four seconds was chosen to make the transition.

Sec. 5.6 MODAL FORCES, DEFLECTIONS, AND DRIFTS

The purpose of this Section is to specify the forces and displacements associated with each of the important modes of response.

Modal forces at each level are given by Formulas 5-4 and 5-4a and are expressed in terms of the gravity load, assigned to the floor, the mode shape and the modal base shear V_m. In applying the forces F_{xm} to the building, the direction of the forces is controlled by the algebraic sign of ϕ_{xm}. Hence, the modal forces for the fundamental mode will all act in the same direction, but modal forces for the second and higher modes will change direction as one moves up the building. The form of Formula 5-4 is somewhat different than usually employed in standard references and shows clearly the relation between the modal forces and the modal base shear. It therefore is a convenient form for calculation and highlights the similarity to Formula 4-6a in the equivalent lateral force procedure.

The modal deflections at each level are specified by Formula 5-5. These are the displacements caused by the modal forces F_{xm} considered as static forces and are representative of the maximum amplitudes of modal response for the essentially elastic motions envisioned within the concept of the seismic response modification coefficient R. This is also a logical point to calculate the modal drifts, which are required in Sec. 5.8. If the mode under consideration were to dominate the earthquake response, the modal deflection under the strongest motion contemplated by the provisions can be estimated by multiplying by the deflection amplification factor C_d. It should be noted also that δ_{xm} is proportional to ϕ_{xm} and will therefore change directions up and down the structure for the higher modes.

Sec. 5.7 MODAL STORY SHEARS AND MOMENTS

This Section merely specifies that the forces of Formula 5-4 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Formula 5-4 are applied to each mass, and linear static methods are used to calculate story shears and story overturning moments. The base shear which results from the calculation should check with Formula 5-1.

Sec. 5.8 DESIGN VALUES

This Section specifies the manner in which the values of story shear, moment, and drift quantities and the deflection at each level are to be combined. The method used, in which the design value is the square root of the sum of the squares of the modal quantities, was selected for its simplicity and its wide familiarity (4,5,7). In general, it gives satisfactory results, but it is not always a conservative predictor of the earthquake response inasmuch as more adverse combinations of modal quantities than are given by this method of combination can happen. The most common instance where combination by use of the square root of the sum of the squares is unconservative occurs when two modes have very nearly the same natural period. In this case the responses are highly correlated and the designer may choose to combine the modal quantities more conservatively (4).

This Section also includes a limit to the reduction of base shear that can be achieved by modal analysis compared to use of the equivalent lateral force procedure. Some reduction, where it occurs, is thought justified because the modal analysis gives a somewhat more accurate representation of the earthquake response. However, it was the intent of these provisions to limit any possible reduction which may occur from the calculation of longer natural periods, because the actual periods may not be as long due to some stiffening effects of nonstructural and architectural components even at moderately large amplitudes of motion. The reduction in base shear is limited to that corresponding to T_1 exceeding T_a by 40 percent.

Sec. 5.9 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

This Section specifies that the design story shears calculated in Sec. 5.8 and the torsional moments prescribed in Sec. 4.4 shall be distributed to the vertical elements of the seismic resisting system as specified in Sec. 4.4 and elaborated upon in the corresponding section of this Commentary. This is consistent with the assumption of planar motion used in this simplified version of modal analysis, and has the intent of providing resistance against torsional response.

However, lateral and torsional motions may be strongly coupled if the building is irregular in its plan configuration (see Sec. 3.4) or if the building, although regular in plan and even with nearly coincident centers of mass and resistance, has its lower natural frequencies nearly equal. The designer should account for the effects of torsion in such buildings in a more accurate manner using methods of modal analysis capable of at least three degrees of freedom per floor (two translational and one torsional). (See Sec. 3.4 of this Commentary.)

Sec. 5.10 FOUNDATION OVERTURNING

Because story moments are calculated mode by mode (properly recognizing that the direction of forces F_{xm} is controlled by the algebraic sign of ϕ_{xm}) and then combined to obtain the design values of story moments, there is no reason for reducing these design moments. This is in contrast with reductions permitted in overturning moments calculated from equivalent lateral forces in the analysis procedures of Chapter 4. (See Sec. 4.5 of this Commentary.) However, in the design of the foundation, the overturning moment calculated at the foundation-soil interface may be reduced by 10 percent, for reasons mentioned in Sec. 4.5 of this Commentary.

Sec. 5.11 P-DELTA EFFECTS

The Commentary for Sec. 4.6 applies to this Section. In addition, to obtain the story drifts when using the Modal Analysis Procedure of Chapter 5, the story drift for each mode shall be independently determined in each story (Sec. 5.8). The story drift shall not be determined from the differential combined lateral building deflections, as this latter procedure will tend to mask the higher mode effects in longer period structures.

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COMMENTARY

CHAPTER 6: SOIL-STRUCTURE INTERACTION

Sec. 6.1 BACKGROUND AND SCOPE

STATEMENT OF PROBLEM: Fundamental to the design provisions presented in Chapters 4 and 5 is the assumption that the motion which is experienced by the base of a structure during an earthquake is the same as the free-field ground motion, a term that refers to the motion which would occur at the level of the foundation if no structure was present. Strictly speaking, this assumption is true only for structures supported on essentially rigid ground. For structures supported on soft soil, the foundation motion is generally different from the free-field motion and may include an important rocking component in addition to a lateral or translational component. The rocking component may be particularly significant for tall structures.

The flexibly supported structure differs from the rigidly supported structure in another important respect: A substantial part of its vibrational energy may be dissipated into the supporting medium by radiation of waves and by hysteretic action in the soil. The importance of the latter factor increases with increasing intensity of ground shaking. There is, of course, no counterpart of this effect of energy dissipation in a rigidly supported structure.

The effects of soil-structure interaction accounted for in this Chapter represent the difference in the response of the structure computed by (1) assuming the motion of the foundation to be the same as the free-field ground motion and (2) considering the modified or actual motion of the foundation. This difference depends on the characteristics of the free-field ground motion as well as on the properties of the structure and the supporting medium.

The interaction effects provided for herein should not be confused with the so-called "site effects". The latter effects refer to the fact that the characteristics of the free-field ground motion induced by a dynamic event at a given site are functions of the properties and geologic features of the subsurface soil and rock. The interaction effects, on the other hand, refer to the fact that the dynamic response of a structure built on that site depends, in addition, on the interrelationship of the structural characteristics and the properties of the local underlying soil deposits. The site effects are reflected in the values of the seismic design coefficients employed in Chapters 4 and 5, and are accounted for only implicitly in this Chapter.

POSSIBLE APPROACHES TO THE PROBLEM: Two different approaches may be used to assess the effects of soil-structure interaction. The first involves modifying the stipulated free-field design ground motion and evaluating the response of the given structure to the modified motion of the foundation, whereas the second involves modifying the dynamic properties of the structure and evaluating the response of the modified structure to the prescribed free-field ground motion (Ref. 36). When properly implemented, both approaches lead to equivalent results. However, the second approach, involving the use of the free-field ground motion, is more convenient for design purposes, and provides the basis of the provisions presented in this Chapter.

CHARACTERISTICS OF INTERACTION: The interaction effects in the approach used herein are expressed by (1) an increase in the fundamental natural period of the structure; and (2) a change (usually an increase) in its effective damping. The increase in period results from the flexibility of the foundation soil, whereas the change in damping results mainly from the effects of energy dissipation in the soil due to radiation and material damping.

These statements are clarified in the following paragraphs by comparing the responses of rigidly and elastically supported systems subjected to a harmonic excitation of the base. Consider the linear structure of weight W, lateral stiffness k, and coefficient of viscous damping c, shown in Fig. C6-1, and assume that it is supported by a foundation of weight W_o at the surface of a homogeneous, elastic halfspace. The foundation mat is idealized as a rigid circular plate of negligible thickness bonded to the supporting medium, and the columns of the structure are considered to be weightless and axially inextensible. Both the foundation weight and the weight of the structure are assumed to be uniformly distributed over circular areas of radius r. The base excitation is specified by the free-field motion of the ground surface. This is taken as a horizontally directed, simple harmonic motion with a period T_o and an acceleration amplitude a_m.

The configuration of this system, which has three degrees of freedom when flexibly supported and a single degree of freedom when fixed at the base, is specified by the lateral displacement and rotation of the foundation, y and θ, and by the displacement relative to the base of the top of the structure, u. The system may be viewed either as the direct model of a one-story building frame or, more generally, as a model of a multistory, multimode structure that responds as a single-degree-of-freedom system in its fixed-base condition. In the latter case, h must be interpreted as the distance from the base to the centroid of the inertia forces associated with the fundamental mode of vibration of the fixed-base structure; and W, k and c must be interpreted as its generalized or effective weight, stiffness, and damping coefficient, respectively. The relevant expressions for these quantities are given in subsequent sections.

The solid lines in Fig. C6-2 and C6-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Fig. C6-1. Two different values of h/r and several different values of the relative flexibility parameter for the soil and the structure, ϕ_o, are considered. The latter parameter is defined by the equation:

$$\phi_o = \frac{h}{v_s T} \tag{C6-1}$$

in which h is the height of the structure, as previously indicated, v_s is the velocity of shear wave propagation in the halfspace, and T is the fixed-base natural period of the structure. A value of $\phi = 0$ corresponds to a rigidly supported structure.

The results in Fig. C6-2 and C6-3 are displayed in a dimensionless form, with the abscissa representing the ratio of the period of the excitation, T_o, to the fixed-base natural period of the system, T, and the ordinate representing the ratio of the amplitude of the actual base shear, V, to the amplitude of the base shear induced in an infinitely stiff, rigidly supported structure. The latter quantity is given by the product ma_m, in which $m = W/g$, g is the acceleration of gravity, and a_m is the acceleration amplitude of the free-field ground motion. The inclined scales on the left represent the deformation amplitude of the superstructure, u, normalized with respect to the displacement amplitude of the free-field ground motion:

$$d_m = \frac{a_m T_o^2}{4\pi^2} \tag{C6-2}$$

The damping of the structure in its fixed-base condition, β, is considered to be 2 percent of the critical value, and the additional parameters needed to characterize completely these solutions are identified in Ref. 37, from which these figures have been reproduced.

Comparison of the results presented in these figures reveals that the effects of soil-structure interaction are most strikingly reflected in a shift of the peak of the response spectrum to the right, and a change in the magnitude of the peak. These changes, which are particularly prominent for the taller structures and the more flexible soils (increasing values of ϕ_o), can conveniently be expressed by an increase in the natural period of the system over its fixed-base value and by a change in its damping factor.

Also shown in these figures in dotted lines are response spectra for single-degree-of-freedom (SDF) oscillators, the natural period and damping of which have been adjusted so that the absolute maximum (resonant) value of the base shear and the associated period are in each case identical to those of the actual interacting systems. The base motion for the replacement oscillator is considered to be the same as the free-field ground motion.

With the properties of the replacement SDF oscillator determined in this manner, it is important to note that the response spectra for the actual and the replacement systems are in excellent agreement over wide ranges of the exciting period on both sides of the resonant peak.

In the context of Fourier analysis, an earthquake motion may be viewed as the result of superposition of harmonic motions of different periods and amplitudes. Inasmuch as the components of the excitation with periods close to the resonant period are likely to be the dominant contributors to the response, the maximum responses of the actual system and of the replacement oscillator can be expected to be in satisfactory agreement for earthquake ground motions as well. This expectation has been confirmed by the results of comprehensive comparative studies that have been carried out (Ref. 36, 37, 38).

It follows that, to the degree of approximation involved in the representation of the actual system by the replacement SDF oscillator, the effects of interaction on maximum response may be expressed by an increase in the fundamental natural period of the fixed-base system and by a change in its damping value. In the following sections, the natural period of the replacement oscillator will be denoted by $\tilde{T}$ and the associated damping factor will be denoted by $\tilde{\beta}$. These quantities will also be referred to as the effective natural period and the effective damping factor of the interacting system. The relationships between $\tilde{T}$ and T and between $\tilde{\beta}$ and β are considered in Sec. 6.2.1(A) and 6.2.1(B).

BASIS OF PROVISIONS AND ASSUMPTIONS: Current knowledge of the effects of soil-structure interaction is derived mainly from studies of systems of the type referred to in the preceding sections, in which the foundation is idealized as a rigid mat. For foundations of this type, both surface-supported and embedded structures resting on uniform as well as layered soil deposits have been investigated (Ref. 4, 7, 13, 16, 24, 29, 36, 37, and 38). However, only a small amount of information is available concerning the interaction effects for structures supported on spread footings or pile foundations (Ref. 6, 21, and 27). The design provisions proposed herein for the latter cases represent the Committee's best interpretation of, and judgement relative to, the current state of knowledge.

Fundamental to the development of these provisions is the assumption that the structure and the underlying soil are bonded and remain so throughout the period of ground shaking. It is further assumed that there is no soil instability or large foundation settlements. The design of the foundation in a manner to ensure satisfactory soil performance, e.g., avoid soil instability and settlement associated with the compaction and liquefaction of loose granular soils, is beyond the scope of this Chapter. Finally, no account is taken of the interaction effects among neighboring structures.

NATURE OF INTERACTION EFFECTS: Depending on the characteristics of the structure and the ground motion under consideration, soil-structure interaction may increase, decrease, or have no effect on the magnitudes of the maximum forces induced in the structure itself (Ref. 4, 13, 36, 37, and 38). However, for the conditions stipulated in the development of the design provisions for rigidly supported structures presented in Chapters 4 and 5, soil-structure interaction will reduce the design values of the base shear and moment from the levels applicable to a rigid-base condition. Therefore, these forces can be evaluated conservatively without the adjustments recommended in this Chapter.

Because of the influence of foundation rocking, however, the horizontal displacements relative to the base of the elastically supported structure may be larger than those of the corresponding fixed-base structure, and this may increase both the required spacing between buildings and the secondary design forces associated with the P-delta effects. Such increases are generally small.

SCOPE: Two procedures are used to incorporate effects of the soil-structure interaction. The first is an extension of the Equivalent Lateral Force Procedure presented in Chapter 4, and involves the use of equivalent lateral static forces. The second is an extension of the simplified Modal Analysis Procedure presented in Chapter 5. In the latter approach, the earthquake-induced effects are expressed as a linear combination of terms, the number of which is equal to the number of stories involved. Other, more complex procedures may also be used, and these are outlined briefly at the end of this Commentary. However, it is believed that the more involved procedures are justified only for unusual buildings of extreme importance and only when the results of the specified simpler approaches have revealed that the interaction effects are indeed of definite consequence in the design.

Sec. 6.2 EQUIVALENT LATERAL FORCE PROCEDURE

This procedure is similar to that used in the current SEAOC provisions except that it incorporates several improvements (see Commentary for Chapter 4). In effect, the procedure considers the response of the structure in its fundamental mode of vibration, and accounts for the contributions of the higher modes implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration. For building structures, this assumption has been found to be adequate (Ref. 5, 13, and 36).

6.2.1 BASE SHEAR

With the effects of soil-structure interaction neglected, the base shear is defined by Formula 4-1:

$$V = C_s W \qquad \text{(4-1) (Chapter 4)}$$

in which W is the total dead weight of the building and of applicable portions of the design live load, as specified in Sec. 4.2; and C_s is the dimensionless seismic design coefficient, defined by Formula 4-2.

The coefficient C_s depends on the seismic zone under consideration, the properties of the site, and the characteristics of the building itself. The latter characteristics include the fixed-base fundamental natural period of the structure, T; the associated damping factor, β; and the degree of permissible inelastic deformation. The damping factor does not appear explicitly in Formula 4-2, because a constant value of $\beta = 0.05$ has been used for all structures for which the interaction effects are negligible. The degree of permissible inelastic action is reflected in the choice of the reduction factor, R.

It is convenient to rewrite Formula 4-1 in the form:

$$V = C_s(T,\beta)\,\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C6-3)}$$

where $\overline{W}$ represents the generalized or effective weight of the structure when vibrating in its fundamental natural mode. The terms in parentheses are used to emphasize the fact that C_s depends upon both T and β. The relationship between $\overline{W}$ and W is given at the end of this subsection. The first term on the right side of Formula C6-3 approximates the contribution of the fundamental mode of vibration, whereas the second term approximates the contributions of the higher natural modes.

Inasmuch as soil-structure interaction may be considered to affect only the contribution of the fundamental mode, and inasmuch as this effect can be expressed by changes in the fundamental natural period and the associated damping of the system, the base shear for the interacting system, $\tilde{V}$, may be stated in a form analogous to Formula C6-3, as follows:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\,\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C6-4)}$$

The value of C_s in the first term of this equation should be evaluated for the natural period and damping of the elastically supported system, $\tilde{T}$ and $\tilde{\beta}$, respectively; and the value of C_s in the second term should be evaluated for the corresponding quantities of the rigidly supported system, T and β.

Before proceeding with the evaluation of the coefficients C_s in Formula C6-4, it is desirable to rewrite this formula in the same form as Formula 6-1 (Chapter 6). On making use of Formula 4-1 and rearranging terms, we obtain the following expression for the reduction in the base shear:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta})\right]\overline{W} \qquad \text{(C6-5)}$$

Within the ranges of natural period and damping that are of interest in studies of building response, the values of C_s corresponding to two different damping values but the same natural period, say $\tilde{T}$, are related approximately as follows:

$$C_s(\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta)\left[\frac{\beta}{\tilde{\beta}}\right]^{0.4} \qquad \text{(C6-6)}$$

This expresssion, which appears to have been first proposed in Ref. 1, is in good agreement with the results of recent studies of earthquake response spectra for systems having different damping values (Ref. 20).

Substitution of Formula C6-6 in Formula C6-5 leads to:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta)\left[\frac{\beta}{\tilde{\beta}}\right]^{0.4}\right]\overline{W} \qquad \text{(C6-7)}$$

where <u>both</u> values of C_s are now for the damping factor of the rigidly supported system, and may be evaluated from Formula 4-2. If the values corresponding to the periods T and $\tilde{T}$ are denoted more simply as C_s and $\tilde{C}_s$, respectively, and if the damping factor β is taken as 0.05, Formula C6-7 reduces to Formula 6-2 (Chapter 6).

It should be noted that $\tilde{C}_s$ in Formula 6-2 is smaller than or equal to C_s, because Formula 4-2 is a non-increasing function of the natural period and $\tilde{T}$ is greater than or equal to T. Furthermore, since the minimum value of $\tilde{\beta}$ is taken as $\tilde{\beta} = \beta = 0.05$ (see statement following Formula 6-9), the shear reduction ΔV is a non-negative quantity. It follows that the design value of the base shear for the elastically supported structure cannot be greater than that for the associated rigid-base structure.

The effective weight of the building, $\overline{W}$, is defined by Formula 5-2 (Chapter 5), in which ϕ_{im} should be interpreted as the displacement amplitude of the ith floor when the structure is vibrating in its fixed-base fundamental natural mode. It should be clear that the ratio $\overline{W}/W$ depends on the detailed characteristics of the structure. A constant value of $\overline{W} = 0.7W$ is recommended in the interest of simplicity and because it is a good approximation for typical buildings. As an example, it is noted that for a tall building for which the weight is uniformly distributed along the height and for which the fundamental natural mode increases linearly from the base to the top, the exact value of $\overline{W} = 0.75\ W$. Naturally, when the full weight of the structure is concentrated at a single level, $\overline{W}$ should be taken equal to W.

The maximum permissible reduction in base shear due to the effects of soil-structure interaction is set at 30 percent of the value calculated for a rigid-base condition. It is expected, however, that this limit will control only infrequently, and that the calculated reduction will in most cases be less.

6.2.1 (A) EFFECTIVE BUILDING PERIOD: Formula 6-3 for the effective natural period of the elastically supported structure, $\tilde{T}$, is determined from analyses in which the superstructure is presumed to respond in its fixed-base fundamental mode and the foundation weight is considered to be negligible in comparison to the weight of the superstructure (Ref. 13 and 37). The first term on the right side of this formula represents the period of the fixed-base structure; the second term represents the contribution to $\tilde{T}$ of the translational flexibility of the foundation; and the third term represents the contribution of the corresponding rocking flexibility. The quantities $\overline{k}$ and $\overline{h}$ represent, respectively, the effective stiffness and effective height of the structure; and K_y and K_θ represent the translational and rocking stiffnesses of the foundation.

Formula 6-4 for the structural stiffness, $\overline{k}$, is deduced from the well-known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{\frac{1}{g}\frac{\overline{W}}{\overline{k}}} \qquad \text{(C6-8)}$$

The effective height, $\overline{h}$, is defined by Formula 6-13, in which ϕ_{i1} has the same meaning as the quantity ϕ_{im} in Formula 5-2 (Chapter 5) when m = 1. In the interest of simplicity and consistency with the approximation used in the definition of $\overline{W}$, however, a constant value of $\overline{h} = 0.7\ h_n$ is recommended, where h_n is the total height of the structure. This value represents a good approximation for typical buildings. As an example, it is noted that for tall buildings for which the fundamental natural mode increases linearly with height, the exact value of $\overline{h} = \frac{2}{3}h_n$. Naturally, when the gravity load of the structure is effectively concentrated at a single level, h_n must be taken equal to the distance from the base to the level of weight concentration.

FOUNDATION STIFFNESSES: These stiffnesses depend on the geometry of the foundation-soil contact area, the properties of the soil beneath the foundation, and the characteristics of the foundation motion. Most of the available information on this subject is derived from analytical studies of the response of harmonically excited rigid circular foundations, and it is desirable to begin with a brief review of these results.

Circular Mat Foundations. For foundations of this type supported at the surface of a homogeneous halfspace, the stiffnesses K_y and K_θ are given by:

$$K_y = \frac{8\alpha_y}{2-\nu}\,Gr \qquad \text{(C6-9)}$$

and

$$K_\theta = \frac{8\alpha_\theta}{3(1-\nu)}\,Gr^3 \qquad \text{(C6-10)}$$

where r is the radius of the foundation; G is the shear modulus of the halfspace; ν is its Poisson's ratio; and α_y and α_θ are dimensionless coefficients that depend on the period of the excitation, the dimensions of the foundation, and the properties of the supporting medium (Ref. 17, 40, and 41). The shear modulus is related to the shear wave velocity, v_s, by the formula:

$$G = \frac{\gamma v_s^2}{g} \tag{C6-11}$$

in which γ is the unit weight of the material. The values of G, v_s, and ν should be interpreted as average values for the region of the soil that is affected by the forces acting on the foundation, and should correspond to the conditions developed during the design earthquake. The evaluation of these quantities is considered further in subsequent sections. For statically loaded foundations, the stiffness coefficients α_y and α_θ are unity, and Formulas C6-9 and C6-10 reduce to:

$$K_y = \frac{8Gr}{2-\nu} \tag{C6-12}$$

$$K_\theta = \frac{8Gr^3}{3(1-\nu)} \tag{C6-13}$$

Studies of the interaction effects in structure-soil systems have shown that, within the ranges of parameters that are of interest for building structures subjected to earthquakes, the results are insensitive to the period-dependency of α_y and α_θ, and that it is sufficiently accurate for practical purposes to use the static stiffnesses, defined by Formulas C6-12 and C6-13.

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas:

$$K_y \approx \frac{8Gr}{2-\nu}\left[1 + \frac{2}{3}\frac{d}{r}\right] \tag{C6-14}$$

$$K_\theta \approx \frac{8Gr^3}{3(1-\nu)}\left[1 + 2\frac{d}{r}\right] \tag{C6-15}$$

in which d is the depth of embedment. These formulas are based on finite element solutions (Ref. 6).

Both analyses and available test data (Ref. 9) indicate that the effects of foundation embedment are sensitive to the condition of the backfill, and that judgement must be exercised in using Formulas C6-14 and C6-15. For example, if a structure is embedded in such a way that there is no positive contact between the soil and the walls of the structure, or when any existing contact cannot reasonably be expected to remain effective during the stipulated design ground motion, then the stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d in Formulas C6-14 and C6-15 should be interpreted as the <u>effective</u> depth of foundation embedment for the conditions that would prevail during the design earthquake.

The formulas for K_y and K_θ presented above are strictly valid only for foundations supported on reasonably uniform soil deposits. When the foundation rests on a stratum of soft soil underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, K_y and K_θ may be determined from the following generalized formulas:

$$K_y \simeq \frac{8Gr}{2-\nu}\left[1+\frac{2}{3}\frac{d}{r}\right]\left[1+\frac{1}{2}\frac{r}{D_s}\right]\left[1+\frac{5}{4}\frac{d}{D_s}\right] \qquad \text{(C6-16)}$$

$$K_\theta \simeq \frac{8Gr^3}{3(1-\nu)}\left[1+2\frac{d}{r}\right]\left[1+\frac{1}{6}\frac{r}{D_s}\right]\left[1+0.7\frac{d}{D_s}\right] \qquad \text{(C6-17)}$$

where G is the shear modulus of the soft soil and D_s is the total depth of the stratum. These formulas are based on analyses of a stratum supported on a rigid base (Ref. 8 and 14).

General Mat Foundations. The information for circular foundations presented in the preceding paragraphs may be applied to mat foundations of arbitrary shapes, provided the following changes are made:

1. The radius r in the expresssions for K_y is replaced by the quantity:

$$r_a = \sqrt{\frac{A_o}{\pi}} \qquad \text{(6-7) (Chapter 6)}$$

which represents the radius of a disk that has the area, A_o, of the actual foundation.

2. The radius r in the expressions for K_θ is replaced by the quantity:

$$r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad \text{(6-8) (Chapter 6)}$$

which represents the radius of a disk that has the moment of inertia, I_o, of the actual foundation.

Footing Foundations. The stiffnesses K_y and K_θ in this case are computed by summing the contributions of the individual footings. If it is assumed that the foundation behaves as a rigid body and that the individual footings are widely spaced so that they act as independent units, then the following formulas are obtained:

$$K_y = \Sigma k_{yi} \qquad \text{(C6-18)}$$

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i} \qquad \text{(C6-19)}$$

The quantity k_{yi} represents the horizontal stiffness of the ith footing; k_{xi} and $k_{\theta i}$ represent, respectively, the corresponding vertical and rocking stiffnesses; and y_i represents the normal distance from the centroid of the ith footing to the rocking axis of the foundation. The summations are considered to extend over all footings. The contribution to K_θ of the rocking stiffnesses of the individual footings, $k_{\theta i}$, is generally small and may be neglected.

The stiffnesses k_{yi}, k_{xi}, and $k_{\theta i}$ are defined by the formulas:

$$k_{yi} = \frac{8G_i r_{ai}}{2-\nu}\left[1+\frac{2}{3}\frac{d_i}{r_{ai}}\right] \qquad \text{(C6-20)}$$

$$K_{xi} = \frac{4G_i\ r_{ai}}{1-\nu}\left[1 + 0.4\frac{d_i}{r_{ai}}\right] \qquad \text{(C6-21)}$$

$$k_{\theta i} = \frac{8G_i\ r_{mi}^3}{2(1-\nu)}\left[1 + 2\frac{d_i}{r_{mi}}\right] \qquad \text{(C6-22)}$$

in which d_i is the depth of effective embedment for the ith footing; G_i is the shear modulus of the soil beneath the ith footing;

$r_{ai} = \sqrt{\frac{A_{oi}}{\pi}}$ = the radius of a circular footing that has the area of the ith footing, A_{oi}; and

$r_{mi} = \sqrt[4]{\frac{4I_{oi}}{\pi}}$ = the radius of a circular footing, the moment of inertia of which about a horizontal centroidal axis is equal to that of the ith footing, I_{oi}, in the direction in which the response is being evaluated.

For surface-supported footings and for embedded footings for which the side wall contact with the soil cannot be considered to be effective during the stipulated design ground motion, d_i in these formulas should be taken as zero. Furthermore, the values of G_i should be consistent with the stress levels expected under the footings, and should be evaluated with due regard for the effects of the dead loads involved. This matter is considered further in subsequent sections.

For closely spaced footings, consideration of the coupling effects among footings will reduce the computed value of the overall foundation stiffness. This reduction will, in turn, increase the fundamental natural period of the system, $\tilde{T}$, and decrease the value of ΔV, the amount by which the base shear is reduced due to soil-structure interaction. It follows that the use of Formulas C6-18 and C6-19 will err on the conservative side in this case. The degree of conservatism involved, however, will partly be compensated by the presence of a basement slab which, even when it is not tied to the structural frame, will increase the overall stiffness of the foundation.

Pile Foundations. The values of K_y and K_θ for pile foundations can be computed in a manner analogous to that described in the preceding section by (1) evaluating the horizontal, vertical, and rocking stiffnesses of the individual piles, k_{yi}, k_{xi}, and $k_{\theta i}$, and (2) combining these stiffnesses in accordance with Formulas C6-18 and C6-19.

The individual pile stiffnesses may be determined from field tests, or analytically by treating each pile as a beam on an elastic subgrade. Numerous formulas are available in the literature (Ref. 19) that express these stiffnesses in terms of the modulus of the subgrade reaction and the properties of the pile itself. Although they differ in appearance, these formulas lead to practically similar results. These stiffnesses are typically expressed in terms of the stiffness of an equivalent free-standing cantilever, the physical properties and cross-sectional dimensions of which are the same as those of the actual pile but the length of which is adjusted appropriately. The effective lengths of the equivalent cantilevers for horizontal motion and for rocking or bending motion are slightly different, but are often assumed to be equal. On the other hand, the effective length in vertical motion is generally considerably greater. For further details, the reader is referred to Ref. 19.

SOIL PROPERTIES: The soil properties of interest are the shear modulus, G, or the associated shear wave velocity, v_s; the unit weight, γ; and Poisson's ratio, ν. These quantities are likely to vary from point to point of a construction site, and it is necessary to use average values for the soil region that is affected by the forces acting on the foundation. The depth of significant influence is a function of the dimensions of the foundation base and of the direction of the motion involved. The effective depth may be considered to extend to about 4 r_a below the foundation base for horizontal and vertical motions, and to about 1.5 r_m for rocking motion. For mat foundations, the effective depth is related to the total plan dimensions of the mat, whereas for buildings supported on widely spaced spread footings, it is related to the dimensions of the individual footings. For closely spaced footings, the effective depth may be determined by superposition of the "pressure bulbs" induced by the forces acting on the individual footings.

Since the stress-strain relations for soils are nonlinear, the values of G and v_s also are functions of the strain levels involved. In the formulas presented in the preceding sections, G should be interpreted as the <u>secant</u> shear modulus corresponding to the significant strain level in the affected region of the foundation soil. The approximate relationship of this modulus to the modulus G_o corresponding to small amplitude strains (of the order of 10^{-3} percent or less) is given in Table 6-A (Chapter 6). The backgrounds of this relationship and of the corresponding relationship for v_s/v_{so} are identified below.

<u>Low-Amplitude Values of G and v_s</u>. The low-amplitude value of the shear modulus, G_o, can most conveniently be determined from the associated value of the shear wave velocity, v_{so}, by use of Formula C6-11. The latter value may be determined approximately from empirical relations, or more accurately by means of field tests or laboratory tests.

The quantities G_o and v_{so} depend on a large number of factors (Ref. 11, 12, and 28), of which the most important is the void ratio, e, and the average confining pressure $\bar{\sigma}_o$. The value of the latter pressure at a given depth beneath a particular building foundation may be expresssed as the sum of two terms as follows:

$$\bar{\sigma}_o = \bar{\sigma}_{os} + \bar{\sigma}_{ob} \qquad \text{(C6-23)}$$

in which $\bar{\sigma}_{os}$ represents the contribution of the weight of the soil, and $\bar{\sigma}_{ob}$ represents the contribution of the superposed weight of the building and foundation. The first term is defined by the formula:

$$\bar{\sigma}_{os} = \frac{1 + 2K_o}{3} \gamma' x \qquad \text{(C6-24)}$$

in which x is the depth of the soil below the ground surface, γ' is the average effective unit weight of the soil to the depth under consideration; and K_o is the coefficient of horizontal earth pressure at rest. For sands and gravel, K_o has a value of 0.5 to 0.6, whereas for soft clays, $K_o \simeq 1.0$. The pressures $\bar{\sigma}_{ob}$ developed by the weight of the building can be estimated from the theory of elasticity (Ref. 25). In contrast to $\bar{\sigma}_{os}$ which increases linearly with depth, the pressures $\bar{\sigma}_{ob}$ decrease with depth. As already noted, the value of v_{so} should correspond to the average value of $\bar{\sigma}_o$ in the region of the soil which is affected by the forces acting on the foundation.

Empirical Relations: For clean sands and gravels having e < 0.80, the low-amplitude shear wave velocity can be calculated approximately from the formula:

$$v_{so} = c_1(2.17 - e)\,(\bar{\sigma}_o)^{0.25} \qquad \text{(C6-25)}$$

in which

c_1 = 78.2 when $\bar{\sigma}_o$ is in lb/ft^2 and v_{so} in ft/sec;

c_1 = 160.4 when $\bar{\sigma}_o$ is in kg/cm^2 and v_{so} in m/sec; and

c_1 = 51.0 when $\bar{\sigma}_o$ is in kN/m^2 and v_{so} in m/sec.

C6.2.1(A) Cont.

For angular-grained cohesionless soils ($e > 0.6$), the following empirical equation may be used:

$$v_{so} = c_2(2.97 - e)(\bar{\sigma})^{0.25} \qquad \text{(C6-26)}$$

in which

$c_2 = 53.2$ when $\bar{\sigma}_o$ is in lb/ft^2 and v_{so} in ft/sec;

$c_2 = 109.7$ when $\bar{\sigma}_o$ is in kg/cm^2 and v_{so} in m/sec; and

$c_2 = 34.9$ when $\bar{\sigma}_o$ is in kN/m^2 and v_{so} in m/sec.

Formula C6-26 may also be used to obtain a first-order estimate of v_{so} for normally consolidated cohesive soils. A crude estimate of the shear modulus, G_o, for such soils may also be obtained from the relationship:

$$G_o = 1{,}000\ S_u \qquad \text{(C6-27)}$$

in which S_u is the shearing strength of the soil as developed in an unconfined compression test. The coefficient 1,000 represents a typical value, which varied from 250 to about 2,500 for tests on different soils (Ref. 10 and 12).

These empirical relations may be used to obtain preliminary, order-of-magnitude estimates. For more accurate evaluations field and /or laboratory determinations may be required.

Field Tests: Field evaluations of the variations of v_{so} throughout the construction site can be carried out by standard seismic refraction methods or by the cross-hole method. The cross-hole method (Ref. 3 and 33) provides information from undisturbed soils below the proposed location of a particular building foundation. The method permits evaluation of v_{so} in layered soils and is not affected by the presence of water in the soil. The low-amplitude procedure is relatively inexpensive and easy to use. The disadvantage of this method is that v_{so} is determined only for the stress conditions existing at the time of the test (usually $\bar{\sigma}_{so}$). The effect of the changes in the stress conditions caused by construction must be considered by use of Formula C6-23 and Formula C6-25 or C6-26 to adjust the field measurement of v_{so} to correspond to the prototype situations. The influence of large-amplitude shearing strains may be evaluated from laboratory tests or approximated through the use of Table 6-A (Chapter 6). This matter is considered further in the next two sections.

Laboratory Tests: Laboratory tests to evaluate v_{so} are usually carried out with resonant column devices (Ref. 28). Such tests may be used to assess the effects of changes in confining pressures, shearing strain amplitudes, stress histories, temperature, and other variables. Consequently, they can easily simulate variations in prototype loading conditions. They are particularly useful in establishing the effects of changes in confining pressures. In fact, Formulas C6-25 and C6-26 were developed from the results of such tests.

<u>Effect of Strain Amplitude on v_s and G</u>. An increase in the shearing strain amplitude is associated with a reduction in the secant shear modulus, G, and the corresponding value of v_s. Extensive laboratory tests (Ref. 2, 12, and 15, for example) have established the magnitudes of the reductions in v_s for both sands and clays as the shearing strain amplitude increases.

The results of such tests form the basis for the information presented in Table 6-A (Chapter 6). For each severity of anticipated ground shaking, represented by the effective peak acceleration coefficients A_a and A_v, a representative value of shearing strain amplitude was developed. Then a conservative value of v_s/v_{so} was established that is appropriate to that strain amplitude. It should be emphasized that the values in this table are first-order approximations. More precise evaluations would require laboratory tests on undisturbed samples from the site, and studies of wave propagation for the site to determine the magnitude of the soil strains induced.

Poisson's Ratio. It is satisfactory to assume Poisson's ratio for soils as:

ν = 0.33 for clean sands and gravels;
ν = 0.40 for stiff clays and cohesive soils; and
ν = 0.45 for soft clays.

The use of an average value of $\nu = 0.4$ also will be adequate for practical purposes.

ALTERNATIVE APPROACH: Formula 6-5 (Chapter 6) for the period $\tilde{T}$ of buildings supported on mat foundations was deduced from Formula 6-3 by making use of Formulas C6-12 and C6-13, with Poisson's ratio taken as $\nu = 0.4$ and with the radius r interpreted as r_a in Formula C6-12 and as r_m in Formula C6-13. For a nearly square foundation, for which $r_a \simeq r_m \simeq r$, Formula 6-5 reduces to:

$$\tilde{T} = T\sqrt{1 + 25\alpha \frac{r\bar{h}}{V_s^2 T^2}\left[1 + 1.12\left(\frac{\bar{h}}{r}\right)^2\right]} \tag{C6-28}$$

The value of the relative weight parameter, α, is likely to be in the neighborhood of 0.15 for typical buildings.

6.2.1 (B) EFFECTIVE DAMPING. Formula 6-9 for the overall damping factor of the elastically supported structure, $\tilde{\beta}$, was determined from analyses of the harmonic response at resonance of simple systems of the type considered in Figures C6-2 and C6-3. The result is an expression of the form (Ref. 4 and 38):

$$\tilde{\beta} = \beta_0 + \frac{\beta}{(\tilde{T}/T)^3} \tag{C6-29}$$

in which β_0 represents the contribution of the foundation damping, considered in greater detail in the following paragraphs, and the second term represents the contribution of the structural damping. The latter damping is assumed to be of the viscous type. Formula 6-9 corresponds to the value of $\beta = 0.05$ used in the development of the response spectra for rigidly supported systems employed in Chapter 4.

The foundation damping factor, β_0, incorporates the effects of energy dissipation in the soil due to the following sources: (1) the radiation of waves away from the foundation, known as radiation or geometric damping; and (2) the hysteretic or inelastic action in the soil, also known as soil material damping. This factor depends on the geometry of the foundation-soil contact area and on the properties of the structure and the underlying soil deposits.

For mat foundations of circular plan that are supported at the surface of reasonably uniform soil deposits, the three most important parameters which affect the value of β_0 are: (1) the ratio $\tilde{T}/T$ of the fundamental natural periods of the elastically supported and the fixed-base structures; (2) the ratio $\bar{h}/r$ of the effective height of the structure to the radius of the foundation; and (3) the damping capacity of the soil. The latter capacity is measured by the dimensionless ratio $\Delta W_s/W_s$, in which ΔW_s is the area of the hysteresis loop in the stress-strain diagram for a soil specimen undergoing harmonic shearing deformation, and W_s is the strain energy stored in a linearly elastic material subjected to the same maximum stress and strain (i.e., the area of the triangle in the stress-strain diagram between the origin and the point of the maximum induced stress and strain). This ratio is a function of the magnitude of the imposed peak strain, increasing with increasing intensity of excitation or level of strain.

The variation of β_o with $\tilde{T}/T$ and $\bar{h}/r$ is given in Figure 6-1 for two levels of excitation. The dashed lines, which are recommended for values of the effective ground acceleration coefficient, A_v, equal to or less than 0.10, correspond to a value of $\Delta W_s/W_s \simeq 0.3$, whereas the solid lines, which are recommended for A_v values equal to or greater than 0.20, correspond to a value of $\Delta W_s/W_s \simeq 1$. These curves are based on the results of extensive parametric studies (Ref. 36-38) and represent average values. For the ranges of parameters that are of interest in practice, however, the dispersion of the results is small.

For mat foundations of arbitrary shape, the quantity r in Figure 6-1 should be interpreted as a characteristic length that is related to the length of the foundation, L_o, in the direction in which the structure is being analyzed. For short, squatty structures for which $\bar{h}/L_o \leq 0.5$, the overall damping of the structure-foundation system is dominated by the translational action of the foundation, and it is reasonable to interpret r as r_a, the radius of a disk that has the same area as that of the actual foundation (see Formula 6-7). On the other hand, for structures with $\bar{h}/L_o \geq 1$, the interaction effects are dominated by the rocking motion of the foundation, and it is reasonable to define r as the radius r_m of a disk whose static moment of inertia about a horizontal centroidal axis is the same as that of the actual foundation normal to the direction in which the structure is being analyzed (see Formula 6-8).

Subject to the qualifications noted in the following section, the curves in Figure 6-1 may also be used for embedded mat foundations and for foundations involving spread footings or piles. In the latter cases, the quantities A_o and I_o in the expressions for the characteristic foundation length, r, should be interpreted as the area and the moment of inertia of the load-carrying foundation.

In the evaluation of the overall damping of the structure-foundation system, no distinction has been made between surface-supported foundations and embedded foundations. Since the effect of embedment is to increase the damping capacity of the foundation (Ref. 4, 22, and 23), and since such an increase is associated with a reduction in the magnitude of the forces induced in the structure, the use of the recommended provisions for embedded structures will err on the conservative side.

There is one additional source of conservatism in the application of the recommended provisions to buildings with embedded foundations. It results from the assumption that the free-field ground motion at the foundation level is independent of the depth of foundation embedment. Actually, there is evidence to the effect that the severity of the free-field excitation decreases with depth (Ref. 32). This reduction is ignored both in Chapter 6 and in the provisions for rigidly supported structures presented in Chapters 4 and 5.

Formulas 6-9 and C6-29, in combination with the information presented in Figure 6-1, may lead to damping factors for the structure-soil system, $\tilde{\beta}$, which are smaller than the structural damping factor, β. However, since the representative value of $\beta = 0.05$ used in the development of the design provisions for rigidly supported structures is based on the results of tests on actual buildings, it reflects the damping of the full structure-soil system, not merely of the component contributed by the superstructure. Therefore, the value of $\tilde{\beta}$ determined from Formula 6-9 should never be taken less than β, and a low bound of $\tilde{\beta} = \beta = 0.05$ has been imposed. The use of values of $\tilde{\beta} > \beta$ is justified by the fact that the experimental values correspond to extremely small-amplitude motions and do not reflect the effects of the higher soil damping capacities corresponding to the large soil strain levels associated with the design ground motions. The effects of the higher soil damping capacities are appropriately reflected in the values of β_o presented in Figure 6-1.

EXCEPTIONS
For foundations involving a soft soil stratum of reasonably uniform properties underlain by a much stiffer, rock-like material with an abrupt increase in stiffness, the radiation damping effects are practically negligible when the natural period of vibration of the stratum in shear,

$$T_s = \frac{4D_s}{v_s}, \tag{C6-30}$$

is smaller than the natural period of the flexibly supported structure, $\tilde{T}$. The quantity D_S in this formula represents the depth of the stratum. It follows that the values of β_0 presented in Figure 6-1 are applicable only when:

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \geq 1 \tag{C6-31}$$

For

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} < 1 \tag{C6-32}$$

the effective value of the foundation damping factor, β_0', is less than β_0, and it is approximated by the second degree parabola defined by Formula 6-10.

For $T_S/\tilde{T} = 1$, Formula 6-10 leads to $\beta_0' = \beta_0$ whereas for $T_S/\tilde{T} = 0$, it leads to $\beta_0' = 0$, a value which clearly does not provide for the effects of material soil damping. It may be expected, therefore, that the computed values of β_0' corresponding to small values of $T_S/\tilde{T}$ will be conservative. The conservatism involved, however, is partly compensated by the requirement that $\tilde{\beta}$ be no less than $\tilde{\beta} = \beta = 0.05$.

6.2.2 and 6.2.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES AND OTHER EFFECTS. The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are generally different. However, the differences are inconsequential for practical purposes, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the provisions for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures.

Formula 6-11 for the lateral floor displacements relative to the base is similar to that specified for rigidly supported structures, except that it includes the contribution of the foundation rotation θ_0. This rotation is defined by the equation

$$\theta_0 = \frac{\tilde{M}_0}{K_\theta} = \frac{\tilde{V}}{V}\frac{M_0}{K_\theta} \tag{C6-33}$$

in which $\tilde{M}_0$ is the overturning moment at the base of the fixed-base structure computed from the modified or reduced seismic forces, and M_0 is the corresponding moment computed from the unmodified forces. The latter moment should not include the reduction permitted in the design of the foundation. The quantity δ_x in Formula 6-11 represents the deflection at level h_x computed in accordance with the provisions of Chapter 4 using the unmodified seismic forces.

Story drifts and P-delta effects should be evaluated as for structures without interaction, using the displacements that include the contribution of the foundation rotation.

Sec. 6.3 MODAL ANALYSIS PROCEDURE

Studies of the dynamic response of elastically supported multi-degree-of-freedom systems (Ref. 5, 7, and 36) reveal that, within the ranges of parameters that are of interest in the design of building structures subjected to earthquakes, soil-structure interaction affects substantially only the response component contributed by the fundamental mode of vibration of the superstructure. In the design provisions proposed in this Section, the interaction effects are considered only in evaluating the contribution of the fundamental structural mode. The contributions of the higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined, as for rigidly supported structures, by taking the square root of the sum of the squares of the maximum modal contributions.

The interaction effects associated with the response in the fundamental structural mode are determined in a manner analogous to that used in the analysis by the Equivalent Lateral Force method, except that the effective weight and effective height of the structure are computed so as to correspond exactly to those of the fundamental natural mode of the fixed-base structure. More specifically, $\overline{W}$ is computed from:

$$\overline{W} = \overline{W}_1 = \frac{(\Sigma w_i \phi_{i1})^2}{\Sigma w_i \phi_{i1}^2} \tag{C6-34}$$

which is the same as Formula 5-2 (Chapter 5), and $\overline{h}$ is computed from Formula 6-13. The quantity ϕ_{i1} in these formulas represents the displacement amplitude of the ith floor level when the structure is vibrating in its fixed-base fundamental natural mode. The structural stiffness, $\overline{k}$, is obtained from Formula 6-4 by taking $\overline{W} = \overline{W}_1$ and using for T the fundamental natural period of the fixed-base structure, T_1. The fundamental natural period of the interacting system, $\tilde{T}_1$, is then computed from Formula 6-3 (or Formula 6-5 when applicable) by taking $T = T_1$. The effective damping in the first mode, $\tilde{\beta}_1$, is determined from Formula 6-9 (and Formula 6-10 when applicable) in combination with the information given in Figure 6-1. The quantity $\overline{h}$ in the latter figure is computed from Formula 6-13.

With the values of $\tilde{T}_1$ and $\tilde{\beta}_1$ established, the reduction in the base shear for the first mode, ΔV_1, is computed from Formula 6-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_1 and $\tilde{T}_1$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_1$; and $\overline{W}$ should be determined from Formula C6-34.

The sections of the recommended provisions on lateral forces, shears, overturning moments, and displacements follow directly from what has already been noted in this and the preceding sections, and need no elaboration. It may only be pointed out that the first term on the right side of Formula 6-15 represents the contribution of the foundation rotation.

OTHER METHODS OF CONSIDERING THE EFFECTS OF SOIL-STRUCTURE INTERACTION

The procedures proposed in the preceding sections for incorporating the effects of soil-structure interaction provide sufficient flexibility and accuracy for practical applications. Only for unusual structures of major importance, and only when the recommended provisions indicate that the interaction effects are of definite consequence in design, would the use of more elaborate procedures be justified.

Following are some of the refinements that are possible, listed in order of more or less increasing complexity:

1. Improve the estimates of the static stiffnesses of the foundation, K_y and K_θ, and of the foundation damping factor, β_o, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil. Solutions may be obtained in some cases with analytical or semi-analytical formulations, and in some cases by application of finite difference or finite element techniques (Ref. 6, 17, 21, and 39). It should be noted, however, that these solutions involve approximations of their own which may offset, at least in part, the apparent increase in accuracy.

2. Improve the estimates of the average properties of the foundation soils for the stipulated design ground motion. This would require both laboratory tests on undisturbed samples from the site and studies of wave propagation for the site. The laboratory tests are needed to establish the actual variations with shearing strain amplitude of the shear modulus and damping capacity of the soil, whereas the wave propagation studies are needed to establish realistic values for the predominant soil strains induced by the design ground motion.

3. Incorporate the effects of interaction for the higher modes of vibration of the structure, either approximately by application of the procedures recommended in Ref. 5, 29, and 34, or by more precise analyses of the structure-soil system. The latter analyses may be implemented either in the frequency domain by use of Fourier transform techniques (Ref. 13 and 36), or directly in the time domain by application of the impulse response functions presented in Ref. 40. However, the frequency domain analysis is limited to systems that respond within the elastic range, while the approach involving the use of the impulse response functions is limited at present to soil deposits that can adequately be represented as a uniform elastic halfspace. The effects of yielding in the structure and/or supporting medium can be considered only approximately in this approach, by representing the supporting medium by a series of springs and dashpots whose properties are independent of the frequency of the motion, and by integrating numerically the governing equations of motion (Ref. 24).

4. Analyze the structure-soil system by the finite element method (Ref. 31, 32, and 35), taking due account of the nonlinear effects in both the structure and the supporting medium.

It should be emphasized that, while they may be appropriate in special cases for design verification, the more elaborate methods referred to above involve their own approximations, and do not eliminate the uncertainties that are inherent in the modeling of the structure-foundation-soil system, and in the specification of the design ground motion and of the properties of the structure and soil.

CONCLUSION

The recommended design provisions are believed to be as simple and rational as they can be made at the present state of knowledge. Furthermore, they are capable of refinement in light of new information without the need for revising the basic approach. The sections of the provisions that stand to benefit most from further research are those concerning buildings supported on piles or spread footings.

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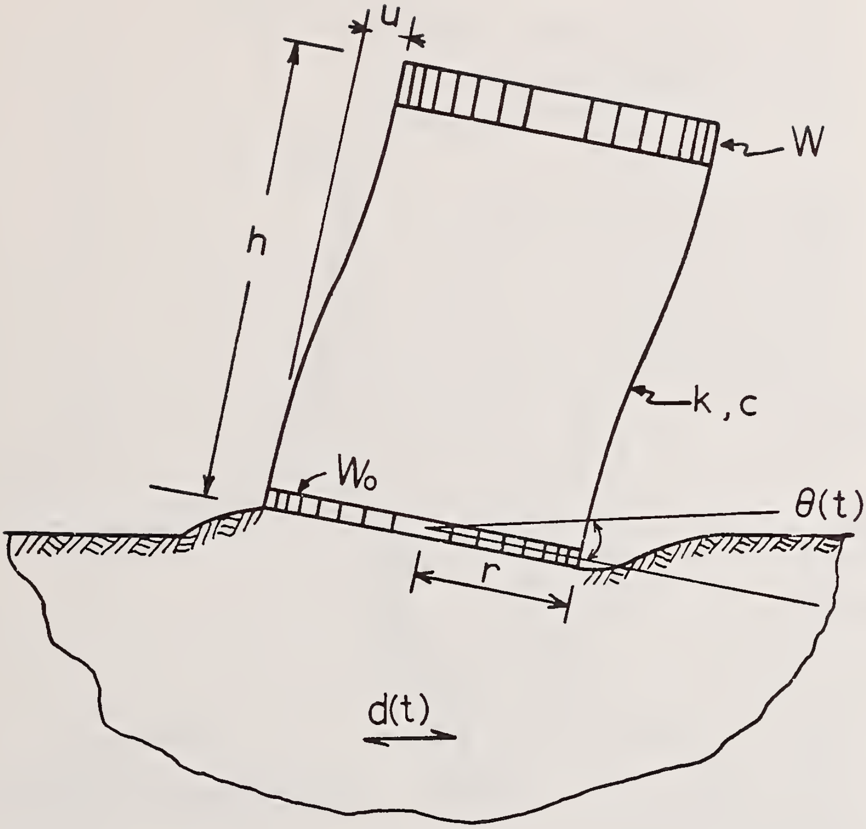

FIGURE C6-1 SIMPLE SYSTEM INVESTIGATED

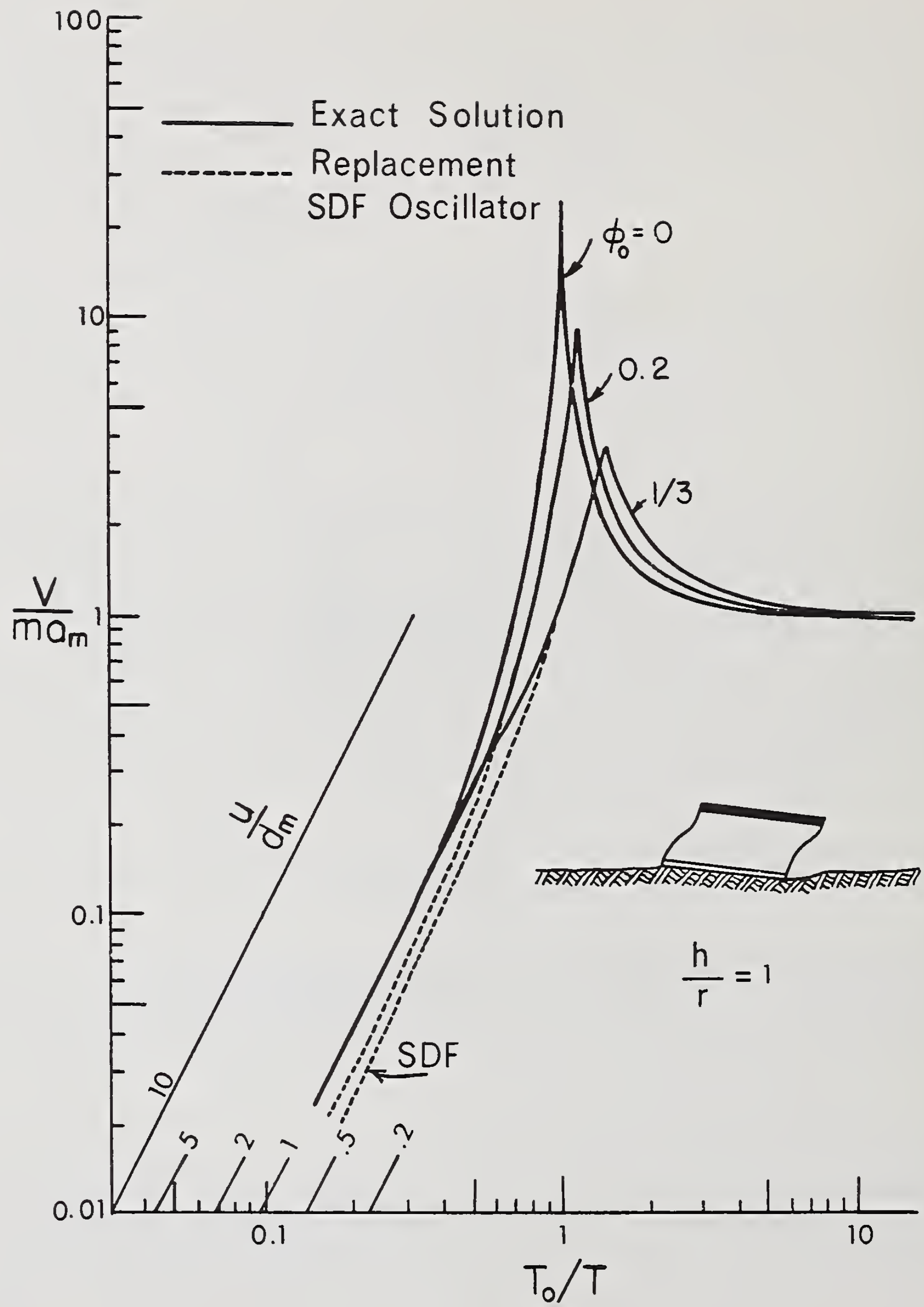

FIGURE C6-2 RESPONSE SPECTRA FOR SYSTEMS WITH h/r = 1

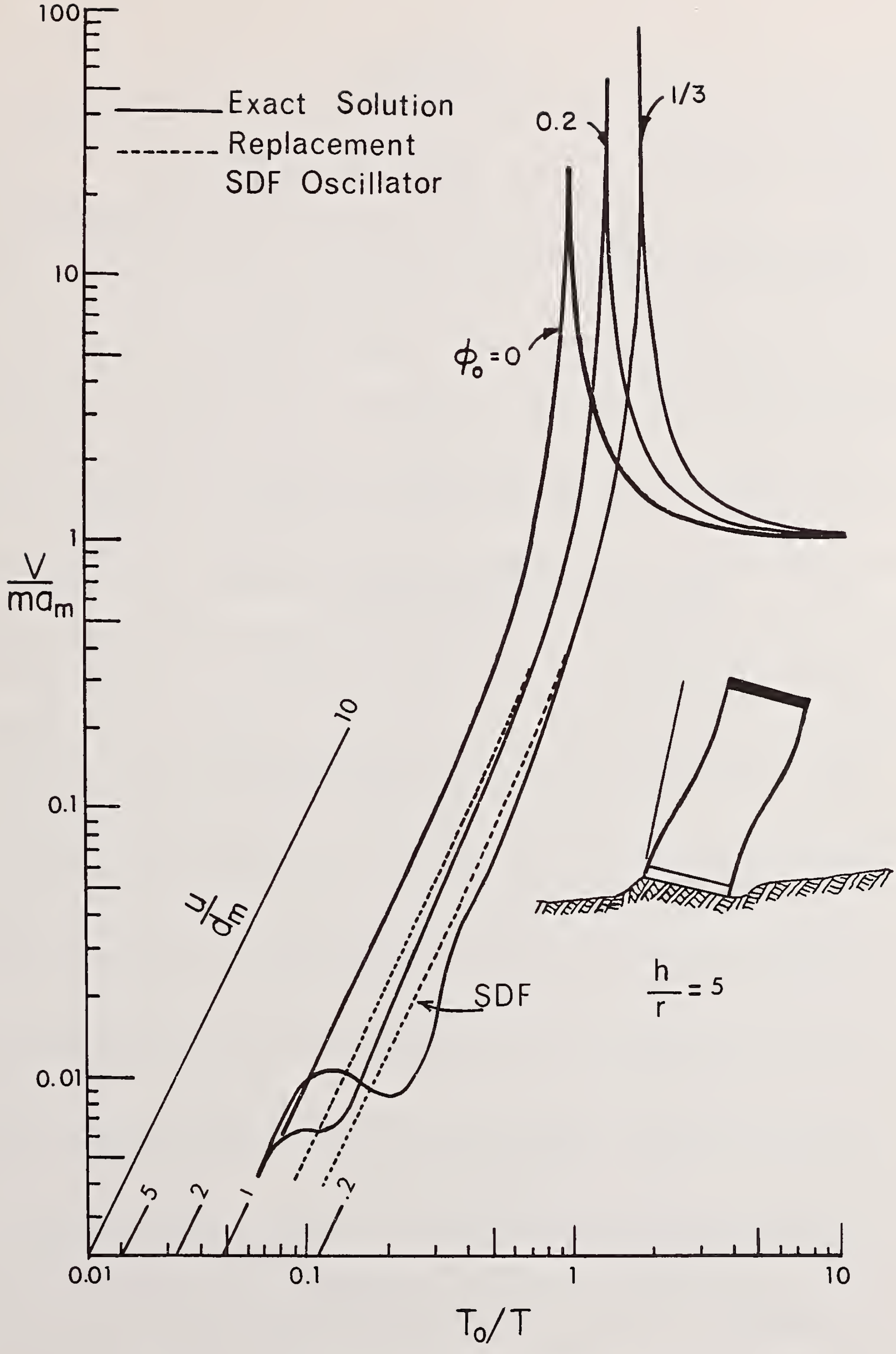

FIGURE C6-3 RESPONSE SPECTRA FOR SYSTEMS WITH h/r = 5

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COMMENTARY

CHAPTER 7: FOUNDATION DESIGN REQUIREMENTS

Sec. 7.1 GENERAL

The minimum foundation design requirements which might be suitable where even minimal consideration must be given to earthquake resistance are set forth in Chapter 7. It is difficult to separate foundation requirements for minimal earthquake resistance from the requirements for resisting normal vertical loads. In order to have a minimum base from which to start, this Chapter assumes compliance with all basic requirements necessary to provide support for vertical loads and lateral loads other than earthquake. These basic requirements include, but are not limited to, provisions for extent of investigation needed to establish criteria for fills, slope stability, expansive soils, allowable soil pressures, footings for specialized construction, drainage, settlement control, and pile requirements and capacities. Certain detail requirements and the allowable stresses to be used are provided in other chapters of the provisions, as well as the additional requirements to be used in more seismically active locations.

Sec. 7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS

The resisting capacities of the foundations shall meet the provisions of this Chapter.

7.2.1 STRUCTURAL MATERIALS

The strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall be as determined in Chapters 9, 10, 11, or 12.

7.2.2 SOIL CAPACITIES

This subsection provides that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil.

Sec. 7.3 SEISMIC PERFORMANCE CATEGORY A

There are no special seismic provisions for the design of foundations for buildings assigned to Category A.

Sec. 7.4 SEISMIC PERFORMANCE CATEGORY B

Extra precautions are required for the seismic design of foundations for buildings assigned to Category B.

7.4.1 INVESTIGATION

The Regulatory Agency may require a formal foundation investigation and a written report. Potential site hazards such as slope instability, liquefaction, and surface rupture due to faulting or lurching as a result of earthquake motions should be investigated when the Regulatory Agency feels the size and importance of the project so warrants or when there may be reason to suspect such potential hazards. Suggested procedures for evaluation of liquefaction potential are given at the end of this Commentary Chapter.

7.4.2 POLE-TYPE STRUCTURES

The use of pole-type structures is permitted.

7.4.3 FOUNDATION TIES

One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation which acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to $A_v/4$ times the larger pile cap or column load.

A common practice in some multistory buildings is to have major columns run the full height of the building and then be separated by smaller columns in the basement which support only the first floor slab. The coefficient applies to the heaviest column load.

Alternate methods of tying foundations together are permitted, such as using a properly reinforced floor slab that can take both tension and compression. Lateral soil pressure on pile caps is not a recommended method because the motion is imparted from soil to structure (not inversely, as commonly assumed); and if the soil is soft enough to require ties, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions.

If the piles are supporting structures in the air or over water (such as in a wharf or pier), batter piles may be required to provide stability or the piles must be designed to provide bending capacity for lateral stability. Hence it is up to the foundation engineer to determine the fluidity or viscosity of the soil to the point where lateral buckling support cannot be provided or where the flow of the soil around the piles may be negligible and provisions for stability are needed. In the ordinary pile-supported building, this is a major reason for the piles and footings to be interconnected so that they act as a unit.

7.4.4 SPECIAL PILE REQUIREMENTS

Special requirements for concrete or composite concrete and steel piles are given in this section. The piles must be connected to the pile caps with dowels.

Whereas unreinforced concrete piles may be in common use in certain areas of the country, their brittle nature when trying to conform to ground deformations makes their use in earthquake-resistant design undesirable. Nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie elements together and to assist in load transfer at the top of pile to the pile cap. Experience has shown that concrete piles tend to hinge or shatter immediately below the pile cap, so tie spacing is reduced in this area to better contain the concrete. In the case of the metal-cased pile, it is assumed that the metal casing provides containment and also a nominal amount of longitudinal reinforcement in the lower portion of the pile.

Bending stresses in piles caused by transfer of seismic motions from ground to structure need not be considered unless the foundation engineer determines that it is necessary. It has been a convenient analytical assumption to assume that earthquake forces originate in the building and are transmitted into and resisted by the ground. Actually the force or motion comes from the ground -- not the structure, as is conveniently assumed for purposes of computation. This makes the necessity of interconnecting footings more important, but what is desired is stability -- not the introduction of forces.

Possibly the simplest illustration is shown in Figure C7-1. Consider a small structure subjected to an external force such as wind; the piles must resist that force in lateral pressure on the lee side of the piles. However, if the structure is forced to move during an earthquake, the wave motion is transmitted through the firmer soils, causing the looser soils at the surface and the building to move. For most structures, the structure weight is negligile in comparison to the weight of the surrounding surface soils. If an unloaded pile were placed in the soil, it would be forced to bend just the same as a pile supporting a building.

The primary requirement is stability, and this is best provided by piles which can support their loads while still conforming to the ground motions, hence the need for ductility.

Sec. 7.5 SEISMIC PERFORMANCE CATEGORY C

For Category C construction, all of the preceding provisions for Categories A and B apply for the foundations, but the earthquake detailing is more severe and demanding. Adequate pile ductility is required and provisions must be made for additional reinforcing to assure, as a minimum, full ductility in the upper portions of the pile.

7.5.1 INVESTIGATION

While the normal pressures on basement walls and retaining walls under normal or static conditions may be assumed to be predictable, the data for loads on walls during earthquakes are meager. Analyses based on the normal assumptions indicate rather high pressures, but general experience in earthquakes indicates that failures have not usually resulted. There is evidence, however, that under some conditions, especially in softer soils, these high pressures may be justified. Consequently, after considering the size and importance of the project and the particular soil conditions, it is left for the foundation engineer to determine the design lateral pressure under dynamic conditins.

7.5.2 FOUNDATION TIES

The additional requirement is made that spread footings should be interconnected by ties. The reasoning given previously under Sec. 7.4.3 applies here also.

7.5.3 SPECIAL PILE REQUIREMENTS

Additional pile reinforcing over that specified for Category B buildings is required. The reasoning given under Sec. 7.4.4 applies here also.

Sec. 7.6 SEISMIC PERFORMANCE CATEGORY D

Foundations for buildings assigned to Category D have one additional requirement over those specified for Category C: precast-prestressed piles shall not be used to resist flexure caused by earthquake motions.

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.

7.4.1 Cont. PROCEDURES FOR EVALUATION OF LIQUEFACTION POTENTIAL

There are basically two methods available for evaluating the cyclic liquefaction potential of a deposit of saturated sand subjected to earthquake shaking:

1. Using methods based on field observations of the performance of sand deposits in previous earthquakes and involving the use of some in-situ characteristic of the deposits to determine probable similarities or dissimilarities between those sites and a proposed new site with regard to their potential behavior

and 2. Using methods based on an evaluation of the cyclic stress conditions likely to be developed in the field by a selected design earthquake and a comparison of these stresses with those observed to cause liquefaction of representative samples of the deposit in some appropriate laboratory test which provides an adequate simulation of field conditions or which can provide results permitting an assessment of the soil behavior under field conditions.

These are often considered to be quite different approaches, since the first method is based on empirical correlations of field conditions and performance, while the second method is based entirely on an analysis of stress conditions and the use of laboratory testing procedures.

In fact, however, because of the manner in which field performance data is usually expressed, the two methods involve the same basic approach and differ only in the manner in which the field liquefaction characteristics are determined.

Thus, for example, it has been found that the most convenient parameter for expressing the cyclic liquefaction characteristics of a sand under level ground conditions is the cyclic stress ratio; that is, the ratio of the average cyclic shear stress τ_h developed on horizontal surfaces of the sand as a result of the cyclic or earthquake loading to the initial vertical effective stress σ_o' acting on the sand layer before the cyclic stresses were applied. This parameter has the advantage of taking into account the depth of the soil layer involved, the depth of the water table and the intensity of earthquake shaking or other cyclic loading phenomenon.

The cyclic stress ratio developed in the field due to earthquake shaking can readily be computed from an equation of the form (Seed and Idress, 1971):

$$\frac{(\tau_h)_{av}}{\sigma_o'} \simeq 0.65 \, \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d \qquad \text{(C7-1)}$$

where a_{max} = maximum acceleration at the ground surface (a value which may be taken to be equal to the effective peak acceleration in any zone).

σ_o = total overburden pressure on sand layer under consolidation

σ_o' = initial effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of 1 at the ground surface to a value of 0.9 at a depth of about 30 ft.

and values of this parameter have been correlated, for sites which have and have not liquefied, with parameters such as relative density based on penetration test data (Seed and Peacock, 1971) or some form of corrected penetration resistance (Seed et al, 1975; Castro, 1975). The latest form of this type of correlation (after Seed, 1976) is shown in Figure 1. In this form of presentation N_1 is the measured penetration resistance of the sand corrected to an effective overburden pressure of 1 ton per sq. ft., based on the results of Gibbs and Holtz (1958) and Bieganousky and Marcuson (1976a and b) using the relationship

$$N_1 = C_N \cdot N \qquad \text{(C7-2)}$$

where C_N is a function of the effective overburden pressure and may be determined from the chart shown in Figure 2 (Seed et al, 1977). Thus for any given site and a given value of maximum ground surface acceleration, the average stress ratio developed during the earthquake $(\tau_h)_{av}/\sigma_o'$ can readily be determined from equation (1) and compared with the value of $(\tau_h)_{av}/\sigma_o'$ at which liquefaction can be expected to occur as determined from Figure 1 for the appropriate magnitude of the earthquake causing ground motions at the site. Use of this procedure may be considered satisfactory for sand deposits to a depth of 40 feet. Alternatively the value of $(\tau_h)_{av}/\sigma_o'$ required to cause liquefaction of the soil at any site may be determined by laboratory tests on samples of the soil involved, the test conditions being chosen to simulate as closely as possible the environmental conditions (soil condition, overburden pressure, etc.) existing in the field.

In utilizing the empirical field correlation approach or the laboratory testing approach, therefore, the procedure followed is the same, differing only on whether the cyclic stress ratio required to cause liquefaction in the field is determined by

1. A correlation between cyclic stress ratios known to have caused liquefaction in previous earthquakes and some significant soil characteristic

or 2. An appropriate laboratory determination of the cyclic stress ratio required to cause cyclic liquefaction of the in-situ deposit. When this procedure is used the appropriate number of stress cycles to be used in the test should be determined

In both cases a factor of safety against liquefaction can be determined by comparing the stress ratio required to cause cyclic liquefaction with that induced by the design earthquake. In general, where field correlations are used a factor of safety of at least 1.5 should be required to establish the safety of a soil against liquefaction, but if detailed laboratory tests are used in conjunction with field data, the factor of safety may be reduced to 1.3.

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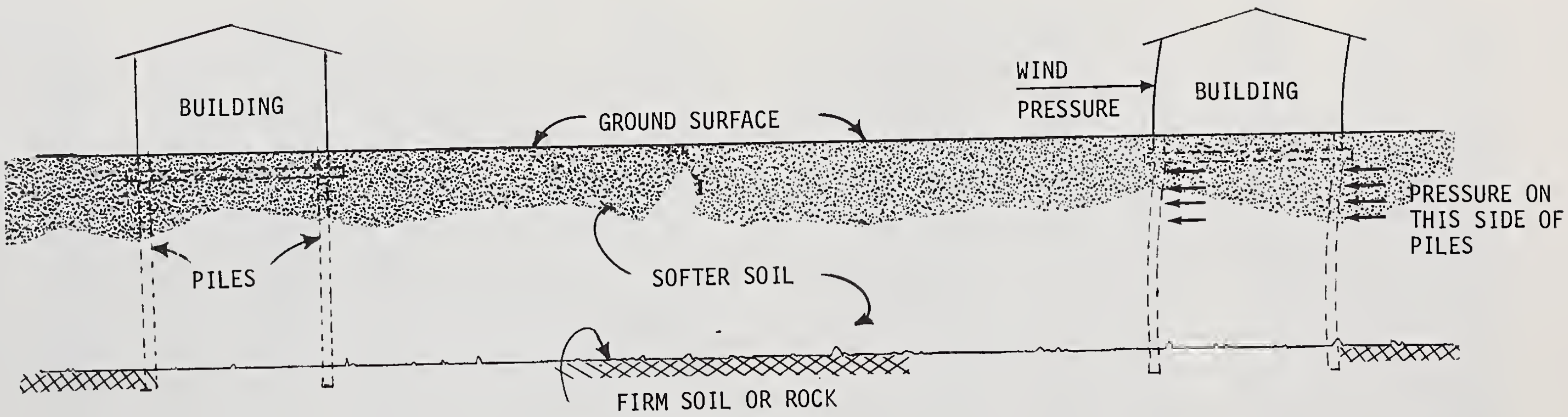
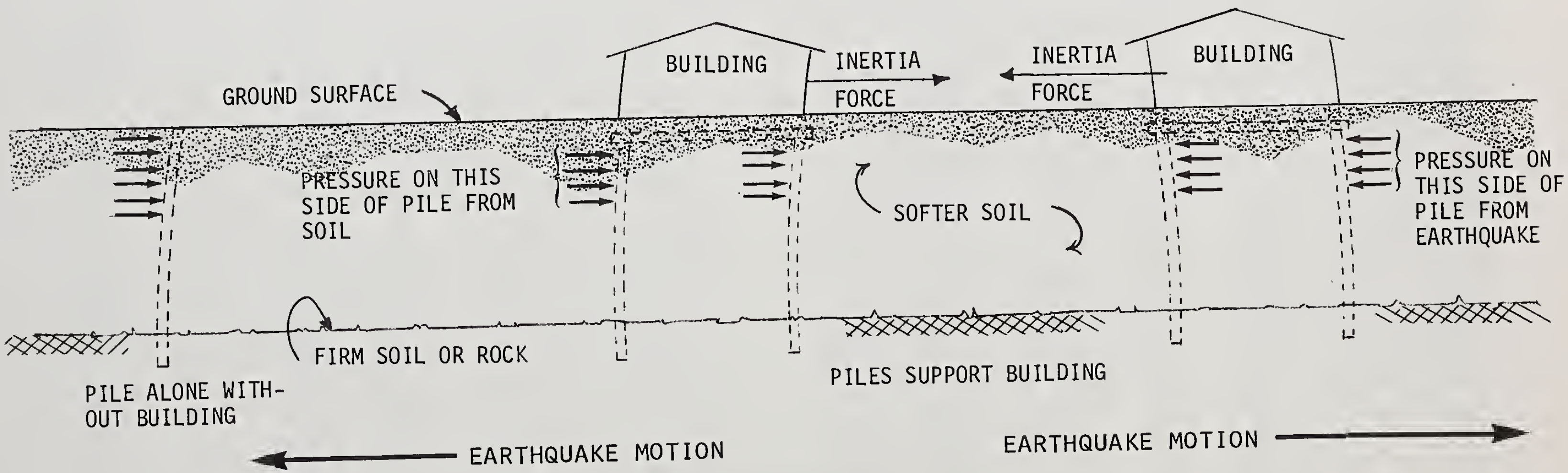


RESPONSE TO WIND



RESPONSE TO EARTHQUAKE

FIGURE C7-1

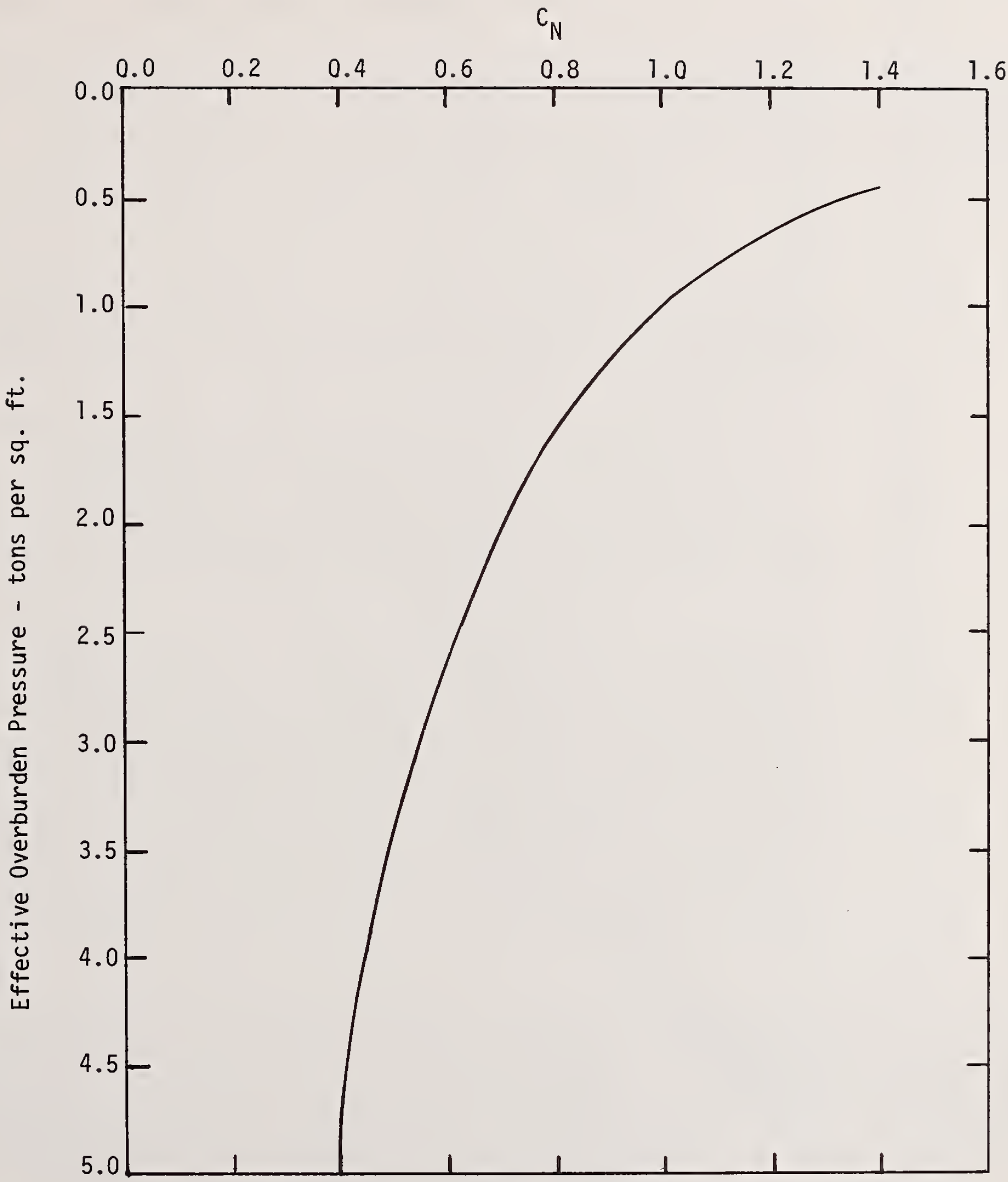


Figure C7-2 RELATIONSHIP BETWEEN C_N AND EFFECTIVE OVERBURDEN PRESSURE

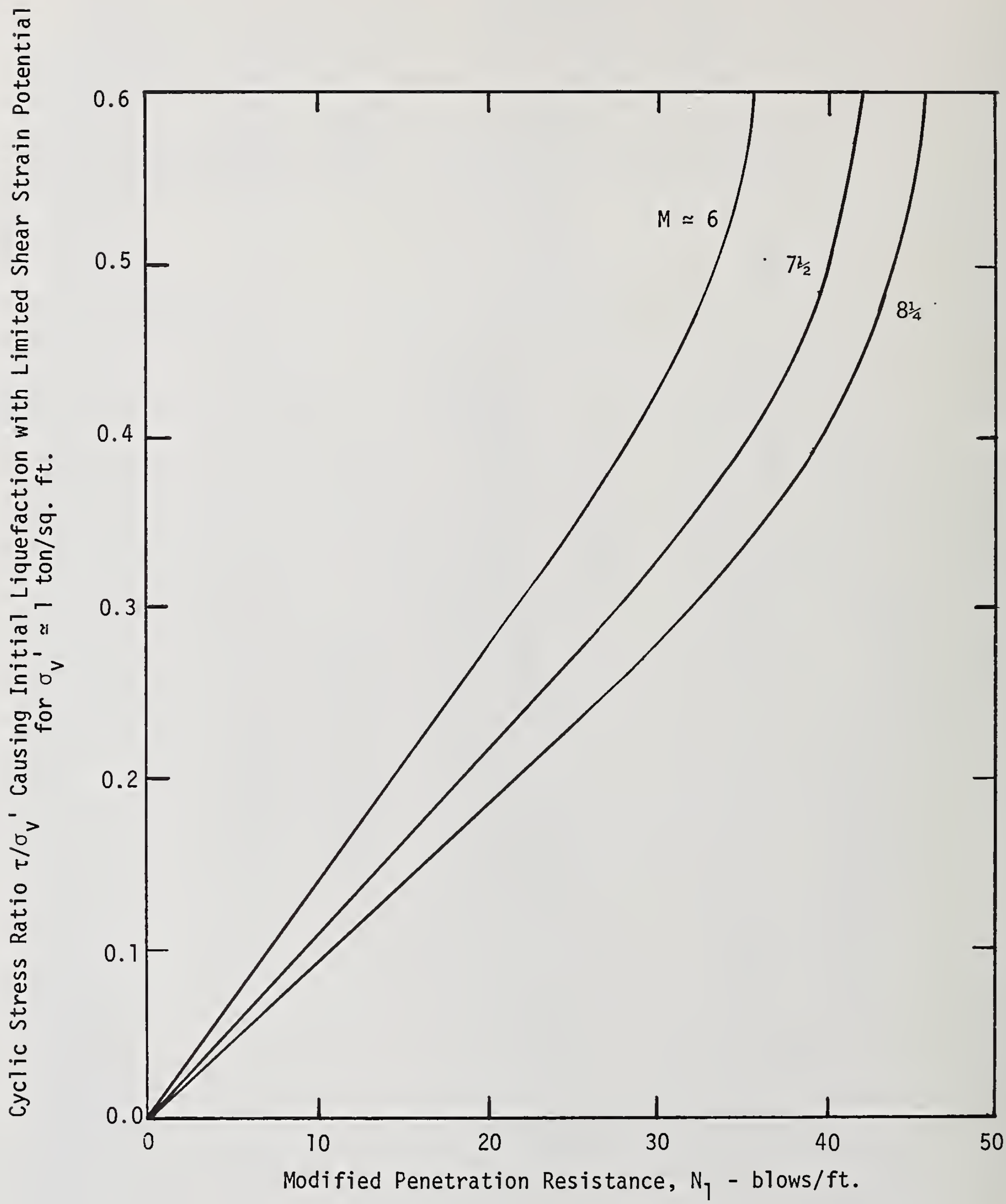


Figure C7-3 CORRELATION BETWEEN FIELD LIQUEFACTION BEHAVIOR OF SANDS FOR LEVEL GROUND CONDITIONS AND PENETRATION RESISTANCE

COMMENTARY

CHAPTER 8: ARCHITECTURAL, MECHANICAL AND ELECTRICAL COMPONENTS

A. BACKGROUND TO ARCHITECTURAL CONSIDERATIONS

The primary intent was to investigate and develop seismic design standards for the performance of the architectural systems and components of a building as they affect life safety. This Commentary discusses the general attitudes and concepts adopted in approaching the subject. Of secondary but still critical importance was the examination of the damage control aspects of those critical facilities most necessary for the survival and recovery of the general public immediately following a major earthquake.

A methodology was devised to relate the following three basic items:

- Architectural components: An orderly classification should be established for architectural components and systems encompassing broad general areas but be definitive enough to give guidance for similar conditions not specifically spelled out or covered.

- Occupancy Classification: Current building code occupancy classifications are based primarily on fire safety and as such do not necessarily or appropriately relate to seismic needs. Accordingly, provisions were developed to relate occupancy classification to the respective hazards of their seismic exposure. See Commentary for Chapter 1, Sec. 1.4.2 (Seismic Hazard Exposure Groups), for detailed explanation.

- Performance Standards: It was deemed desirable to develop performance standards and not rely on mathematical coefficients as has been the norm in standards of this type. For example, the design of a suspended ceiling in a hospital should have a higher level of performance capability than the same system in a warehouse in order to provide for life safety and maintenance of operability. On the other hand for certain systems or components such as exterior wall panels the concern for life safety requires similar performance of the system regardless of the occupancy involved. However, this objective could not be fulfilled and the end result is similar to the traditional approach using numerical factors.

The objective was to study the effects of seismically-induced forces and deformations on the nonstructural (specifically, architectural) components in all types of building uses. Appropriate guidelines and design provisions for architectural systems and components were to be developed from a life safety standpoint. Each architectural component was to be examined as a function of expected performance, building occupancy and function, and its placement or location as a component of the building system. Finally, consideration was to be given to the architectural planning and design process as a means of improving the man-built environment from a life-safety standpoint relative to seismic hazards.

The building designer has a responsibility to consider the relative levels of damage experienced by a building during an earthquake. These levels are a direct function of:

- The architectural concept as expressed by the design of the building.

- The resistance of the materials of construction.

- The intensity of the ground motion.

The initial overall architectural concept has a direct bearing on the seismic resistance of a building and a considerable effect on the potential mitigation of hazards resulting from seismic forces. For the architect certain principles and responsibilities hold just as true in designing systems and components for earthquake-resistant buildings as in the creation of any functional object. The designer, in addition to conceiving a rational design concept of the total building for seismic loading, must articulate all components into a logical system integrated as a unit rather than as a series of unconnected parts.

Architectural systems may be affected directly by the seismic forces or indirectly by interaction with the structural framing system or other architectural or mechanical and electrical systems. Fabrication methods used to connect the component parts to the structure or to each other are therefore as critical as the preliminary design. Connection details require specific attention since a dislodged roofing tile unit falling from a building could be as lethal to an individual as the failure of a primary girder. The life safety aspect of falling building debris associated with earthquake damage is related to a series of variables which include:

- Relationship of the location of the earthquake with respect to densely populated urban centers.
- The time of day (number of people in the area).
- The design and construction characteristics of the building occupied by or immediately adjacent to people.

Depending on the time of day and the resultant amount of activity without and within the building, falling debris from the building may cause as great a number of casualties to pedestrians or motorists as to building occupants. It was with such potential exterior hazards in mind that the City of Los Angeles enacted a "parapet ordinance" in 1949 which requires the strengthening or removal of hazardous parapets and appendages to buildings. The potential hazard was demonstrated during the 1971 San Fernando earthquake when the only fatality in the City of Los Angeles occurred when a pedestrian was struck by debris from a collapsing parapet of an old building in downtown Los Angeles approximately 20 miles from San Fernando.

B. BACKGROUND TO MECHANICAL AND ELECTRICAL CONSIDERATIONS.

The objective was to develop seismic criteria for the design and construction of mechanical and electrical systems and equipment and their attachments to the building structure so as to increase the protection of life and public welfare. A secondary objective was to define an acceptable level of damage. In so doing, consideration was given to the occupancy and function of the building.

Traditionally, mechanical and electrical systems for buildings have been designed with little, if any, regard to stability when subjected to seismic forces. Exceptions are to be found in nuclear power plant design and other special-purpose and high-risk structures. Equipment supports have been generally designed for gravity loads only, and attachments to the structure itself were often deliberately designed to be flexible to allow for vibration isolation or thermal expansion.

Few building codes, even in regions with a history of seismic activity, have contained provisions governing the behavior of mechanical and electrical systems. One of the earliest references to seismic bracing can be found in NFPA Pamphlet 13, "Sprinkler Systems". This pamphlet has been updated periodically since 1876, and seismic bracing requirements have been included since about 1940. Until recently little data was available regarding damage to mechanical and electrical equipment. Reports on the Alaskan earthquake of March 17, 1964 and the San Fernando earthquake of February 9, 1971 (Ayres et al 1964, Sharpe et al 1972, and Ayres et al 1972) document damage to mechanical and electrical systems and highlight the problem. These reports indicate that buildings which sustained only minor structural damage became uninhabitable and hazardous to life due to failures of mechanical and electrical systems.

As a result of the San Fernando earthquake, in 1972 legislation was passed in California (SB 519, 1972) establishing seismic criteria for the construction of health care facilities. This bill, which was in essence an extension of the California Field Act (California State Education Code) to health care facilities, included for the first time seismic requirements for mechanical and electrical equipment and systems. The resulting regulations (California Administrative Code) apply to all health care facilities constructed in the State of California after April 1, 1974. The basic philosophy underlying the intent of the law is that the facilities must "be completely functional to perform all necessary services to the public after a disaster" (SB 519, 1972). The regulations require that mechanical and electrical systems be anchored so as to remain in place and be designed to remain operable after an earthquake. Another example of a code that was changed to include requirements for mechanical and electrical equipment is the April 1973 edition of the U.S. Department of Defense Tri-Service Seismic Design Manual (1973). This document was used in the development of the amplification factor used in the provisions of Chapter 8.

In assessing the level of "acceptable damage", secondary effects were considered to a limited extent. Fires and explosions resulting from damaged mechanical and electrical equipment represent secondary effects of earthquakes; these were not considered, however, except as covered under Sec. 8.3.5. Further, the potential danger of secondary damage from falling architectural and structural components, (which could inflict major damage to adjacent equipment and render it unusable) should be carefully assessed by building designers.

These secondary effects can represent a considerable hazard to the building, its occupants, and its contents. Steam and hot water boilers and other pressure vessels can release fluids at hazardous temperatures. Hot water boilers operating above 212°F in particular represent a hazard as the sudden decrease in pressure caused by a rupture of the vessel can result in instantaneous conversion of superheated hot water to steam, with explosive disintegration of the remainder of the vessel. Mechanical systems often include piping systems filled with flammable, toxic, or noxious substances such as ammonia or other refrigerants. Some of the nontoxic halogen refrigerants used in air conditioning apparatus can be converted to a poisonous gas (phosgene) upon contact with open flame. Hot parts of disintegrating boilers, such as portions of the burner, firebrick, etc., are at high enough temperatures to ignite combustible materials with which they might come in contact.

It was concluded that, while secondary effects should eventually be included in building regulations, the provisions of Chapter 8 represent a sufficiently drastic departure from current design practices and the inclusion of secondary effects should be left for the future development of seismic code provisions. This basic philosophy underlies much of the assignment of performance levels to different occupancies.

C. DESIGN CONSIDERATIONS

Four aspects of seismic safety were considered as follows:

- General life safety
- Property damage affecting life safety

- Functional impairment of critical facilities affecting post-disaster recovery (loss of utilities, elevators, life safety elements, etc.)

- Safety of emergency personnel such as fire and rescue teams

The above four criteria objectives are closely interrelated because property damage resulting from the consequences of an earthquake can be a definite cause of life loss. As in the case of fire, the relative hazards to life safety are also directly related to the occupancy load and the actual use of the building. The greater the occupancy load, the greater the potential life loss during an earthquake. An unoccupied building does not present a hazard to life safety within the structure during an earthquake.

Earthquake damage studies have shown that the placement of nonstructural elements on or in a building may have significant effects in modifying the seismic response of the structure. Heretofore this aspect of building design has received little attention. For example, prior seismic design philosophy implied that little structural damage should occur during moderate ground motion but some damage was expected to nonstructural components of the building. This inferred that, while the possibility of structural collapse may be minimal, there was little concern in design for earthquake-induced forces acting upon architectural and other nonstructural components. Recent earthquakes have demonstrated that the cost of damage to such components can be excessive.

Four sources of forces were considered with regard to the nonstructural components or systems:

- Seismic induced forces acting directly on the component or system

- Seismic induced forces acting directly on the component or system joints or connections

- Seismic induced deformation of the structural frame generating forces acting directly on the component or system

- Seismic induced deformation of the structural frame generating forces acting directly on the component or system joints or connections

D. SCOPE

In the development of the provisions, it was necessary to analyze all nonstructural components for consequences to life safety and building function. Initially, all architectural components of a building were considered and those determined inconsequential to life safety were excluded. The remainder were assessed as to their potential effect on people and expected performance. Table C8-1 gives a complete list of all the architectural components and systems considered. It represents most of the architectural components of a building which could present hazardous exposure to the public. Similar listings were prepared for the mechanical and electrical components and systems. Initial consideration was given to 172 individual mechanical and electrical components in 37 occupancy classifications, in an effort to arrive at common characteristics. Subsequently, these were consolidated, resulting in 19 component groups in the three seismic hazard exposure groups listed in Table 8-C. It was recognized that not all buildings contain all the components listed. The list represents a fairly complete compilation of components and systems some or all of which are usually present in typical or atypical buildings. Practical considerations--most notably enforcement--resulted in the modification, consolidation, and reduction in the number and type of components subject to seismic design requirements as specified in Tables 8-B and 8-C. It is assumed that the building designers will work as a team to provide for the required performance levels.

Sec. 8.1 GENERAL REQUIREMENTS AND 8.1.1 INTERRELATIONSHIPS OF COMPONENTS

The general requirements establish minimum design levels for architectural, mechanical, and electrical systems and components recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical and electrical components.

EXCEPTIONS:

1. Those systems or components designated in Table 8-B or 8-C for L performance level which are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with a Seismicity Index of 1 or 2 or which are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with a Seismicity Index of 1 are not subject to the provisions of this Chapter.
2. Where alterations or repairs are made the forces on systems or components in existing buildings may be modified in accordance with the provisions of Sec. 13.3.

Seismic Hazard Exposure Groups are determined in Sec. 1.4. Mixed Occupancy requirements are provided in that Section.

The seismic force on any component shall be applied at the center of gravity of the component and shall be assumed to act in any horizontal direction. For vertical forces on mechanical and electrical components, see Table 8-C, Footnote 2.

Although the components and systems listed in Tables 8-B and 8-C are presented separately, significant interrelationships exist between them. For example, exterior, nonstructural, spandrel walls may shatter and fall on the streets or walks below, seriously hampering accessibility and egress functions. Further the rupture of one component could lead to the failure of another which is dependent on the first. Accordingly, the collapse of a single component may ultimately lead to the failure of an entire system. Widespread collapse of suspended ceilings and light fixtures in a building may render an important space or major exit stairway unusable. Such types of interrelationships exist for the components in Tables 8-B and 8-C and should not be overlooked.

Consideration was also given to the design requirements for these components to determine how well they are conceived for their intended functions. Potential beneficial and/or detrimental interactions with the structure were examined. The interrelationship between components or systems and their attachments were surveyed. Attention was given to the performance relative to each other of architectural, mechanical and electrical components, building products and finish materials, and systems within and without the building structure. It should be noted that the modification of one component in Table 8-B or 8-C could affect another, and in some cases such a modification could help reduce the risk associated with the interrelated unit. For example, landscaping barriers around the exterior of certain buildings could diminish the risk due to falling debris, although this should not be interpreted to mean all buildings must be landscaped.

The design of systems or components that are in contact with or in close proximity to other structural or nonstructural systems or components must be given special study to avoid damage or failure when seismic motion occurs. If a ceiling supports a wall, the intersection must be detailed to accommodate differential movements between them. Another example is where an important element of a system, such as a motor-generator unit for a hospital is adjacent to a nonload-bearing partition. The failure of the partition might jeopardize the motor-generator unit and therefore the wall should be designed for a performance level sufficient to assure its stability.

Where nonstructural wall systems may affect or stiffen the structural system because of their close proximity, care must be exercised in selecting the wall materials and in designing the intersection details to assure the desired performance of each system.

8.1.2 CONNECTIONS AND ATTACHMENTS

It is required that the components be attached to the building structure and that all the required connections and attachments be fully detailed in the design documents. These details should take into account the force levels and anticipated deformations expected or designed into the system. See also Sec. 8.2.3.

If an architectural component or system were to fail during an earthquake, the mode of failure would probably be related to:

- Faulty design of the component
- Interrelationship with another component which fails
- Interaction with the structural framing system
- Deficiencies in its type of mounting
- Inadequacy of its connections or anchorage

The last is perhaps the most critical when considering seismic safety.

Building components designed without any intended structural function--such as in-filled walls--may interact with the structural framing system and be forced to act structurally as a result of excessive building deformation. The build up of stress at the connecting surfaces or joints may exceed the limits of the materials. Spatial tolerances between such components thus becomes a governing factor. Therefore the provisions place emphasis on the ductility and strength of the connections for exterior wall elements and the interrelationship of elements.

Traditionally, mechanical equipment which does not include rotating or reciprocating components (tanks, heat exchangers, etc.) is rigidly anchored to the building structure. Mechanical and electrical equipment containing rotating or reciprocating components is often isolated from the structure by vibration isolators (rubber-in-shear, springs, air cushions). Heavy mechanical equipment (such as large boilers) is often not restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, is usually rigidly anchored (switchgear, motor control centers, etc.). The installation of unattached mechanical and electrical equipment should be virtually eliminated for buildings covered by the provisions.

Friction cannot be counted on to resist seismic forces because it has been observed that equipment and fixtures often tend to "walk" due to rocking when subjected to earthquake motions. This is often accentuated by the vertical ground motions. Because frictional resistance cannot be relied upon; positive restraint must be provided for each system or component.

8.1.3 PERFORMANCE CRITERIA

Each type of component or system subject to these provisions was evaluated as to its expected performance level. The goal of designing for several performance levels, which was established for initial guidance, is contained in Table C8-2. Levels of expected performance were assessed against levels of potential hazards to life safety according to the location and function of the component. Life safety was the overriding criterion for developing the levels of performance for each nonstructural component.

Once a performance criteria is established for a component or system, it should be designed to operate or function at that level. Specifically, performance criteria are utilized to define standards against which expected performance is to be measured in terms of life safety.

The performance characteristic levels, P, given in Table 8-A resulted from consideration of a combination of factors including performance and value judgement based on personal experience. In the development of the "P" values, the formulas utilizing this factor are based on broad assumptions. Therefore, the differences in performance levels are sizable. It should be noted that 1.0 is considered the base performance value for most components.

The factor, P, is a dimensionless modifier of the design force level on a component or system based upon its interrelationship with Seismic Hazard Exposure Group (occupancy or use group) for the building in which it is located. These are shown in Tables 8-B and 8-C.

Sec. 8.2 ARCHITECTURAL DESIGN REQUIREMENTS

8.2.1 GENERAL

The architectural design requirements provide that calculations, criteria or other substantiation be prepared and included as part of the design documentation. The use of standard designs for certain building components, based upon conservative values for variables, may be applicable to most buildings.

The location of a building is important from three viewpoints:

- Site-related effects of ground shaking, including landslide and liquefaction
- Relationship to densely populated areas
- Linkage to site plan

Location and geographic distribution of buildings have a direct relationship to potential life loss. In areas of high-intensity ground shaking the possibility of significant failure of architectural and other nonstructural systems increases. While hazard to life safety within a building remains constant, potential life loss can be significantly increased if the building is also located in a densely populated urban area. The time of day can also be of importance because of the possibility of a large number of persons being inside or adjacent to the exterior of the building.

The placement of buildings on a site can significantly affect the impact that collapse, or failure, of architectural and nonstructural components can have on:

- The entrance or egress of occupants to the building
- The blocking of streets
- Accessibility to the building by fire and rescue teams

Accordingly, guidelines were established to cover the respective hazards and their relationships to both interior occupancies and exterior circulation.

Many variables exist in building linkages to the site plan. Perhaps the most obvious constraint is the effect of lot size and/or location. Few options exist for either the architect or engineer to position buildings on small lots or restricted sites in congested urban centers. However, in the case of large building sites, such as those found in regional shopping complexes surrounded by large parking areas, hazard mitigation can be properly considered. For example, as noted previously, properly placed landscaping around the exterior of a building can provide a protective barrier from falling hazards. Accessibility to a damaged building for fire and rescue teams is essential and therefore the entrance and egress to the building should be protected. All space surrounding a building does not necessarily affect accessibility--only those areas which are associated with accessibility to the building site and entrance to and egress from each building.

Accessibility for Group III occupancies is most important. Experience has shown that access can be lost or seriously compromised from debris falling from both the building involved and adjacent property. In order to assure that future improvements on adjacent property would not jeopardize this accessibility, the provisions require that adequate protection of such access be provided. The simplest means of resolving this adjacent property criteria would be to restrict the location of the access to at least 10 feet from any adjacent property line. If there is an existing building on the adjacent property and it is, for example, constructed of unreinforced masonry, the architect should seriously consider providing a greater degree of protected access. This would avoid the potential hazard that the existing adjacent structure may present. Though not covered by these provisions, the designer should also consider the possible loss of access along streets, highways, or bridges adjacent to the site.

8.2.2 FORCES

The design seismic force is dependent upon the weight of the system or component, the seismic coefficient for the locality, the seismic coefficient for the component, and the required performance characteristic. The term "A_v" is a variable parameter dependent on local earthquake history and probability of occurrence. The maps in Chapter 1 specify values for locations across the United States (U.S.). The performance characteristic relates to the occupancy group and the component or system involved per Table 8-B.

Certain design requirements for architectural components in areas of low seismicity are eliminated by the "Exceptions" of this Section. However, the designer may wish to provide for some increased safeguards in order to lessen the potential cost to to his client for architectural components. This is not mandated in the provisions.

It should be noted that the minimum lateral design force usually specified for interior partitions--i.e., the 5 pounds per square foot criteria found in most codes--may exceed the forces developed from Formula 8-1, thereby eliminating the need for seismic design of these walls.

The C_c factor in Table 8-B was originally based on the use of the working stress design and was similar to the C_p factors specified in the Uniform Building Code and Title 24 of the State of California Administrative Code. In some cases these values were modified slightly based upon experience and judgment. In the case of exterior nonbearing walls (parapets), the C_c value was considerably reduced as the committee could not justify a difference between a parapet and a cantilever portion of an exterior wall. The poor history of unreinforced masonry parapets, which was the basis of prior high C_c values, should not be transferred to newer and properly designed systems.

When the decision was made to use stresses approaching yield in the provisions, the C_c values were modified so as to be in accordance with these higher allowable stresses. the final proposed C_c factors (and existing code C_p factors) are somewhat arbitrary and consequently need continued review and further research. It is hoped that future investigations will distinguish between a failure to meet the requirements of a standard and a failure based on noncompliance with the basic intent of a standard and thereby develop more rational values for these factors.

The modifications which resulted in the C_c values presented in Table 8-B were developed from comparative computations and application of subjective judgment:

From prior codes

$$F'_p = ZC'_p W_p \qquad (1)$$

Formula 8-1

$$F_p = A_v C_c P W_c \qquad (2)$$

where

F'_p = the force at working stress level

F_p = the force at yield

Z = the seismic zone factor

A_v = the effective peak velocity-related acceleration coefficient

C'_p = the prior component factor

C_c = the new component factor

W_p & W_c = the weight of component

P = the performance factor

assuming

$Z = 1$, $A_v = 0.4$ for Seismicity Index 4 and $P = 1$,

then

$$F_p = 1.2 F'_p$$

$$F_p = 1.2 \times C'_p W_p \qquad (1)$$

$$F_p = 0.4 \times C_c \times W_c \qquad (2)$$

if

$C'_p = 0.2$ for a partition

then

$$1.2 \times 0.2 \times W_p = 0.4 C_c W_c \qquad (1) \& (2)$$

and

$$C_c = 0.6$$

The amplification effects due to height in a building were not considered significant because of the manner in which the values were assigned to C_c and P, the general relatively small weight of components or systems, (as compared to the building weight), and the desire to maintain a simple form for Formula 8-1.

8.2.3 EXTERIOR WALL PANEL ATTACHMENT

This Section requires ductility and rotational capacity for exterior panels. To ensure that the connection is ductile, care must be taken in detailing the attachments. To minimize the possibility of a brittle-type failure, the connections to the structural frame must be designed to accommodate (by bending or rotation) the potential differential motions between the component and the structural frame.

8.2.4 COMPONENT DEFORMATION

Earthquake motions induce deflections at each floor level. The difference in the deflections of the top and bottom of each story is the story "drift". Walls, partitions, glazing, etc., in each story of a building must be capable of accommodating the story drift without causing a life safety hazard. The larger story drifts resulting from the inherently more-flexible steel or reinforced concrete moment frame buildings may cause damage to floor-to-floor partitions and other nonstructural systems such as stairs, elevator shafts, etc., unless proper design considerations are provided. Such nonstructural damage as evidenced in past earthquakes can exceed 50 percent of the replacement value of a building and can also endanger the occupants. In comparison, shear wall buildings are usually more rigid than moment frame structures and therefore have smaller story drifts. Architectural design considerations must take into account the components of deformation which can occur:

- Direct deformation in the component or system itself
- Direct deformation in the joints or connections of the component or systems
- Deformation of the component or system produced by structural frame or structural wall movements
- Deformation in the joints or connections of the component or system produced by structural frame or structural wall movements

The drift values to be considered in the design of components are those derived in Sec. 4.6.1. These values can be reduced by one-half for components with a required performance characteristic level of L.

All architectural systems or components connected to or framed within the structural system must be capable of accommodating a story drift of Δ without failure or should be separated from the structure to prevent the deformations of the structure from affecting the architectural system or component. Such isolation can be accomplished by providing a degree of separation at least equal to the calculated drift from Sec. 4.6.1. Rigid elements, such as stairways or masonry walls, should be given special consideration since not only are they subject to damage and loss of function from structural deformations but also, of equal importance, their stiffness may significantly affect the structural system to which they are connected. In each instance both structural and fire resistance requirements have to be reconciled.

Differential vertical movement between horizontal cantilevers in adjacent stories-i.e., cantilevered floor slabs-has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

8.2.5 OUT-OF-PLANE BENDING

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

Sec. 8.3 MECHANICAL AND ELECTRICAL DESIGN REQUIREMENTS

8.3.1 GENERAL

The mechanical and electrical design forces are assumed to be imposed from any horizontal direction. The vertical forces as noted in Footnote 1 of Table 8-C are assumed to be one-third of the maximum horizontal forces. The designer is allowed an option of justifying a reduction in the seismic forces required by this Chapter. Such justification may be made by performing a dynamic analysis based upon established principles of structural dynamics.

8.3.2 FORCES

Formula 8-2 shall be used for the design of components and their attachments. The method of attachment for mechanical and electrical components shall be either by fixed or direct attachment to the building or by attachment with a resilient mounting system. Reliance on friction to resist seismic forces is not permitted.

If an item of mechanical or electrical equipment is rigidly anchored to the building structure, seismic forces are transmitted directly to the equipment. The design force is dependent on the performance rating assigned to the particular piece of equipment.

Where fixed (rigid) attachments are used for components with performance levels of S or G in areas with Seismicity Indices of 3 or 4, certification must be obtained from the component manufacturer that the component is capable of withstanding the design forces without sustaining damage. Shaking table tests or three-dimensional shock tests may be used for certification if an analysis is too difficult to perform. Components can frequently withstand considerable force in one horizontal direction, but may fail if a concurrent force is applied from another horizontal direction.

Mechanical equipment such as reciprocating or rotating machinery has traditionally been mounted on resilient mounting systems, particularly when installed on upper floors of structures. The primary reason for this type of mounting system is to dampen or isolate the vibration emanating from the equipment and thereby inhibit sound and vibration transmission through the building structure.

The structural system and the resilient mounted equipment form a complex dynamic system. To account for this, the amplification introduced by the relationship of the equipment support period and the building period should be included if the equipment is to survive the earthquake as required for S or G performance criteria levels. It is recognized that a rigorous solution of this problem requires a detailed computer-type dynamic analysis. The designer is given the option of making a rigorous dynamic analysis of the equipment and its supporting system by established principles of structural dynamics to qualify the equipment. As an alternate, the Tri-Service Seismic Design Manual includes a method based on an approximation of the system as a single-degree-of-freedom system. This method was adapted to the general methodology followed in these provisions as one method of qualifying the equipment. An attempt was made to determine whether techniques are available at present to conduct a meaningful dynamic analysis of elastic restraining systems. The state-of-the-art appears to be as follows:

- Only one commercially-available computer program is known to be available, which provides a form of dynamic analysis of elastic restraining systems. Because of the absence of actual earthquake data, this program makes assumptions regarding frequency components and their duration and limits itself to frequencies in the range of 0.1 to 16 Hz. The program was developed by the California Institute of Technology for a manufacturer of resilient support systems and access is available only through that manufacturer.

- There are sensors and recording systems available which can measure and record direct on magnetic tape the various parameters during a seismic event. The data could form the basis for improved dynamic analysis programs and make possible improved design techniques for resilient mounting systems.

- There is a need for the installation of full dynamic response sensors at existing strong motion instrumentation stations. There is also a need for the development of adequate computer programs which can be made available to all qualified designers in this field.

The resilient mounting attachments shall be designed to decelerate movement of the component or system at a rate which will not generate forces in excess of those calculated from Formula 8-2. The resilient mounting systems can include such items as stable springs, pneumatic restraining devices or elastic restraining devices; however, any device used must be capable of withstanding the forces determined from Formula 8-2.

It was the opinion of the ATC Committee that the equation for calculating the seismic forces on mechanical and electrical equipment should include two variable parameters in addition to those required in Sec. 8.2. Therefore two additional factors; a_c (an amplification factor for resiliently mounted equipment) and a_x (an amplification factor to increase the applied forces dependent on the height of the equipment in the building) are included in Formula 8-2. The values of the various factors and coefficients were determined as follows:

- <u>C_c Factor Determinations</u>. Initially, C_c was defined as $C_c = a/g$

where

g = acceleration due to gravity (ft/sec^2)

a = estimated design acceleration (ft/sec^2)

The quantity "a" represented an amplification of the effective peak acceleration coefficient for Seismicity Index 4. The amount of amplification was related to similar factors in the California regulations resulting from Senate Bill 519. In order to bring C_c into conformance with other sections of the provisions, the concept was changed to define C_c as a numerical dimensionless factor related to the mechanical and electrical components in Table 8-C. The numerical values as shown in Table 8-C were developed by using an analogy to the C_p values in Table T17-23-3 of Title 24, as follows:

Title 24

$$Fp' = Cp' \, Wp \, P$$

where

Fp' = The design force

Cp' = The Cp value from Table T17-23-3

Wp = Weight of component

Formula 8-2

$$Fp = A_v\, C_c\, a_c\, a_x\, W_c\, P$$

where

F_p = The design force

A_v = Effective Peak Velocity-Related Acceleration Coefficient (EPV)

a_x = 1 (for comparison purposes)

a_c = 1 (for comparison purposes)

W_c = Weight of component

P = 1.5 (for a hospital)

F_p was set equal to 1.2 Fp' because the design in these provisions is based on yield strength and not on working strength as in Title 24. Thus:

$$A_v C_c a_x a_c W_c P = 1.2Cp'W_p$$

Substituting $A_v = 0.40$: $0.4C_c 1.5 = 1.2C_p'$

or $C_c = 2.0C_p'$

Table T17-23-3 prescribes $C_p' = 1.0$ for essential mechanical equipment, and thus $C_c = 2.0$ for comparable mechanical and electrical components with an S performance level. Values for other equipment were then scaled to the above.

- Structure Amplification Factor (a_x). The use of the building amplification factor a_x required similar considerations to those above. A review of the literature (Tri-Service Manual, Fagel et al 1973) as well as a desire to motivate designers to locate heavy mechanical or electrical equipment in the lower levels of the building, has prompted the use of such a factor. One method of accounting for this effect is to use a formula based on the distribution factor C_{vx} from Formula 4-6a. The of this formula requires cross-referencing to Chapter 4, and involves concepts which may be unfamiliar to mechanical and electrical engineers. In addition, it tends to result in values in excess of those considered reasonable. Therefore, it was decided to use an approach derived from information contained in the Tri-Services Manual, but which differs from it as follows:

  The equation used in the Tri-Services Manual gives directly the acceleration due to seismic forces (as a fraction of gravity) at each level of the building. This number is then combined with a soil constant such that the product of the structure amplification factor and the soils constant (A_hC_s) represents a number comparable to the product of the EPV coefficient A_v; the C_c factor; structure amplification factor ($A_v \times C_c \times a_x$).

It was judged that a 100 percent increase for the top level of the building was reasonable.

- Equipment Amplification Factor (a_c). A relationship for determining this amplification factor was developed by assuming that the response of the building at the equipment level can be approximated by a sinusoidal loading of the form P sin(ωt). The amplification factor for this type of motion is then related to the acceleration resulting from the increase in the equipment response due to the building response. Whenever the period of the building and that of the equipment are approximately equal, resonance occurs. The equation is based on the theory of harmonic motion (Timoshenko 1955) and is used to compute the amplification factor.

$$a_c = 1/\sqrt{[1-(\omega/\omega_a)^2]^2 + [2\lambda\omega/\omega_a]^2}$$

where

a_c = the amplification factor

ω = the natural frequency of the equipment (rad/sec).

ω_a = the natural frequency of the structure (rad/sec).

λ = the percent of critical damping of equipment.

The Tri-Service Manual has selected a value of λ equal to 2 percent. Substitution of the value $2\pi/T$ for ω, and $2\pi/T_c$ for ω_a produces the curve shown on Plate C8-1 which indicates a magnification factor of 25 at resonance. This was reduced to a factor of 2 for period ratios between 0.6 and 1.4 seconds with all other period ratios having a factor of 1 for the following reasons:

1. The damping coefficient λ is not constant at 2 percent during a seismic event.

2. The building period is also not a constant because of deformation of the structure.

3. The Tri-Service magnification factor graphs are based upon an approximation of the system as a single-degree-of-freedom type system. This is not considered to be representative of actual conditions. It should be noted however that period ratios in the range of 0.8 to 1.2 may result in considerably higher magnification and this must be considered in the design.

4. Component Attachment Period (T_c). Formula 8-4 is derived from the basic mass response equation (Tri Service Manual).

$$\omega = \sqrt{K/M_{me}}$$

where

ω = The circular frequency (rad/sec).

M_{me} = The mass of mechanical or electrical equipment ($lb\text{-}sec^2/in$)

and the period $T = 2\pi/\omega$(seconds)

Combining the above equations

$$T = 2\pi\sqrt{W/K_g}$$

where

g = The acceleration due to gravity ($in./sec^2$)

W = The weight of equipment (lb)

Formula 8-4 results after substituting $2\pi/\sqrt{g} = 0.32$.

8.3.5 UTILITY AND SERVICE INTERFACES

Special hazards to the building and its occupants are created by the failure of utility systems. It was felt necessary to give some consideration to secondary effects of a seismic event, as an exception to the general rule followed elsewhere. Possible secondary effects are leakage of fossil fuels from broken lines, or electrical short-circuit currents in excess of normal protective device capabilities. For this reason, for Group I and Group II Seismic Hazard Exposure Groups in areas with Seismicity Indices of 3 and 4, protective devices are required which will automatically stop fuel flows or interrupt currents in the event earthquake motions greater than a designated intensity occur. Interruption of gas or high temperature energy supplies to buildings can be accomplished by installing seismic valves at the service connection to a building. Interruption of electrical service can be achieved by shunt-tripping the main circuit breakers when activated by a sensor which can detect excessive ground motion.

The ATC Committee also expressed concern regarding the rapid growth of urban electric distribution networks. In many instances utility companies have increased their distribution networks such that the fault current potentials that existed when a building was originally constructed have increased manyfold. This is particularly the case in urban areas where secondary network concepts are utilized. These networks, by adding transformer capacity, have reduced the reactance needed to limit fault current. In some cases, electrical facilities initially providing less than 25,000 amperes interrupting current now exceed 200,000 amperes or more, and incoming service equipment and distribution equipment within the structure are inadequate to handle such loads. This problem is of concern because phase-to-phase or phase-to-ground faults can develop during a seismic event in equipment not adequately designed, which could completely consume the service entrance equipment, service protection equipment, and distribution equipment and represent a significant source of fire. The potential energy release of these fault currents is such that 1/4" x 4" cross-section bus bars, utilized in switchboards singly or in multiples, would melt as if in an electric arc furnace, and the molten copper would flow along the floor, igniting any combustible material it encountered. The resolution of this problem is not within the scope of these provisions.

For essential facilities, equipment and systems requiring an S Performance Characteristics Level must remain in operation after the disaster. For this reason, auxiliary on-site mechanical and electrical utility sources, or secondary utility sources, are recommended. No reference to this situation is included in the provisions because in most cases existing building regulations usually contain such provisions. It is recommended that an appropriate clause be included if the existing codes for the jurisdiction do not presently provide for it.

TABLES 8-B and 8-C OCCUPANCY-COMPONENTS-PERFORMANCE RELATIONSHIPS

The definitions of Architectural Components and Systems (Table C8-1), Occupancy Group Types (Table C8-4), and Criteria for Performance Standards (Table C8-2) have been discussed earlier. It is apparent that interrelationships exist between the items listed in each table which have direct impact on the levels of life safety to be achieved. For example, a heavy piece of mechanical equipment, ceiling mounted, presents minimal hazard to life safety when located in the occupancy group "Private Garages", whereas the hazard from such equipment increases significantly if it is located in a large hall for public assembly with a potential occupancy of more than 1,000. The hazard would be further increased if the connection or mounting for the equipment was poorly designed. An additional increase in the hazard potential would occur if it was mounted on the ceiling of a hospital ward used 24 hours a day. As described earlier the introduction of landscaped barriers may alter the life safety risk from falling objects. Accordingly, design trade-offs between variables could raise or lower the life safety hazard. Following this principle the methodology for dealing with a set of variables was established.

Some critical variables affecting life safety that were used in this methodology are:

- Occupancy density.
- Building height
- The need for functioning after an earthquake considering:

  1. Overall occupancy critical use factor.
  2. Specific component use factor.
  3. Need for egress after an earthquake.
  4. Need for functionability of fire protection.

- Adequate access for emergency personnel.
- Public hazard exposure outside the building.
- Critical exposure to major secondary hazards such as fire, explosion.
- Familiarity of occupants with surroundings.
- Restriction on movement of occupants.
- Probable age and mobility of occupants.
- Siting of building.

Table C8-5 displays the initial results of the methodology when applied to measurement of the three basic variables. It presents these results in the form of a table labeled "Tentative Matrix". The variables are measured against each other and are subject to modification when other sets of variables are introduced. Application of the "Tentative Matrix" to any one architectural component and system correlates the element (subject to further modification if desired) to performance standards and occupancy group. Other patterns may be found by seeking relationships between the architectural component and its performance to occupancy group, or occupancy group and architectural component to performance standard. Thus for most desired information the "Tentative Matrix" display could be utilized to obtain correlation with performance standards, architectural element definition, or occupancy group type. The higher the performance standard displayed on the "Tentative Matrix", the higher the hazard posed by the architectural element in context with occupancy group characteristics. In this way, minimum force levels were developed. The purpose of including this initial table is to provide guidance for future considerations and for evaluation of the method used.

It was therefore clearly evident that a system needed to be devised to measure all variables and establish priorities in dealing with them. Any system so devised had to recognize the interrelationships between all items and correlate their diverse characteristics.

CONCLUDING COMMENTS

1. MAINTENANCE. Mechanical and electrical devices installed to satisfy the requirements of these provisions such as resilient mounting systems or certain protecting devices, require maintenance to ensure their reliability and provide the protection for which they are designed in case of a seismic event. Specifically, rubber-in-shear mounts or spring mounts (if exposed to weathering) will deteriorate with time and thus periodic testing is required to ensure that their damping action will be available during an earthquake. Pneumatic mounting devices and electric switchgear must be maintained free of dirt and corrosion. How a Regulatory Agency could administer such periodic inspections was not determined and hence provisions to cover this situation have not been included.

2. MINIMUM STANDARDS. Criteria represented in the provisions represent minimum standards. They are designed to minimize hazard for occupants and to permit, insofar as practicable, the continued functioning of facilities required by the community to deal with the consequences of a disaster. They are not designed to protect the owner's investment, and the designer of the facility should review with the owner the possibility of exceeding these minimum standards so as to limit his economic risk.

The risk is particularly acute in the case of sealed, air-conditioned buildings with L performance levels where downtime after a disaster can be materially affected by the availability of parts and labor. The parts availability may be significantly worse than normal because of a sudden increase in demand. Skilled labor may also be in short demand, as available labor forces may be diverted to high priority structures requiring repairs.

3. ARCHITECT-ENGINEER DESIGN INTEGRATION. The subject of a architect-engineer design integration is being raised because it is believed that all members of the profession should clearly understand that Chapter 8 is a compromise based on concerns for enforcement and the need to develop within the available time frame a simple, straightforward approach. It is imperative from the outset that architectural input concerning definition of occupancy classification and the required level of seismic resistance be properly integrated with the approach of the structural engineer to seismic safety if the design profession as a whole is to make any meaningful impact on the public conscience in this issue. Accordingly, considerable effort was spent in this area of concern. It is hoped that as the design profession gains more knowledge and sophistication in the use of seismic design, it will collectively be able to develop a more comprehensive approach to earthquake design provisions.

TABLE C8-1

ARCHITECTURAL COMPONENTS AND SYSTEMS SUBJECT TO LIFE SAFETY CONSIDERATIONS

Architectural Component or System Designation

Building Accessibility (includes ground floor egress)
Exterior Non-Structural Walls (includes parapets, large scale veneers)
Veneers, small scale ceramic mosaics, Venetian tile, etc.
Canopies (except as means of egress)
Roofing Units (tile, metal panels, slate, etc.)
Containerized and Miscellaneous Elements (planter boxes, etc.)

Fire Detection systems
Fire Suppression sytems
Life Safety Communications systems
Smoke Removal systems

Stairs
Elevators (operation only)
Vertical Shafts including elevator shafts

Horizontal Exits (only where otherwise required)
Public Corridors
Private Corridors

Full Height Area and Separation Partitions
Full Height Structural Fireproofing
Full Height Other Partitions - Screens Included
Partial Height Partitions - Screens Included

Ceilings - Fire Membrane
Ceilings - Non-Fire Membrane

Equipment - Ceiling Mounted
Equipment - Wall Mounted
Equipment - Free-standing Unstable
Equipment - Free-standing Stable

Furniture - Unstable
Furniture - Stable

Art Work - Ceiling Mounted
Art Work - Wall Mounted
Art Work - Free-standing Unstable
Art Work - Free-standing Stable

TABLE C8-2A

PERFORMANCE CRITERIA FOR ARCHITECTURAL COMPONENTS AND SYSTEMS

Matrix Letter Symbol	Ranking Performance Level Number	Performance Characteristic	Design Goal
S	1	Superior	Maximum resistance to lateral force design criteria. Design limited to cosmetic damages. All operating functions to be unimpaired. Minimize glass breakage (safety glass may crack). No loss of any fire rating or protection. System or component shall be able to handle 1.5 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural member design movement.
G	2	Good	Average resistance to lateral force design criteria. No major fall-off of wall or ceiling components allowed. No glass fall-out except for tempered glass fragments. All operating functions normally operable or readily repaired on-site _______ working days. Fire ratings 75% intact. This does not mean 75% of unit is intact; it means that a 4-hour wall shall have 3-hour, etc. Minor damage to system or component structure allowed. System or component shall be able to handle 1.0 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural design movement.

(TABLE C8-2A CONT.)

Matrix Letter Symbol	Ranking Performance Level Number	Performance Characteristic	Design Goal
L	3	Low	Low resistance to lateral force design criteria. Glass fall-out permitted. Ceilings and lighting fixtures may fall down. Major components must substantially stay in place but not operable until repaired. System or component structural damage may occur. Fire ratings impaired. System or component shall be able to handle 0.5 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural member design movement.
N	4	None	No performance standards required.

TABLE C8-2B

PERFORMANCE CRITERIA FOR MECHANICAL/ELECTRICAL COMPONENTS AND SYSTEMS

Performance Criteria Factor	Performance Level	Design Goal
1.5	Superior (S)	a) High resistance to static and dynamic seismic forces.
		b) All operating functions unimpaired.
		c) No broken piping regardless of size.
		d) No interruptions of utility services other than normal transfer functions to alternate sources.
1.0	Good (G)	a) Moderate resistance to static and dynamic forces.
		b) All major equipment normally operable or easily repaired at site.
		c) No broken main distributing piping or vessel.
		d) No shorted or broken electrical circuits.
0.5	Low (L)	a) Low resistance to static and dynamic seismic forces.
		b) Major equipment must substantially stay in place.
		c) Broken main distribution piping and vessels tolerated.
		d) Fallout of lighting fixtures tolerated.
		e) Shorts or broken electrical circuits tolerated.
0.0	None (N)	No performance standards required.

It should be noted that the design goals listed above do not represent absolute levels. The complexity of mechanical and electrical equipment, piping and duct systems, electrical distribution systems, etc., together with the unique magnitude and time spectrum characteristics of each seismic event make this impossible. It is believed that the above design goals are achievable and that equipment and systems designed to this proposal will result in an acceptable minimum percentage of failures and danger to the public.

TABLE C8-3

INITIAL GENERAL GROUPING OF OCCUPANCIES

Group Letter	General Classification	Sub-Group Code No.	Occupancy Description
A	Typical Public Assembly		Load of 100 or more, includes drinking and dining establishments.
B	Special Public Assembly	1	Open air only (not covered by roof): Stadiums, Reviewing Stands, Park Structures, etc.
		2	Regional Shopping Centers with Enclosed Shopping Malls.
C	Education (Campus operations only; does not include 1 to 3 room adult school operation)	1	50 or more persons through 12th grade.
		2	Less than 50 persons through 12th grade.
D	Confined Facilities	1	Mental, jails, prisons, restrained inmates.
		2	Nuseries for child care only, non-ambulatory.
		3	Nursing homes, child care of kindergarten age or over, ambulatory.
		4	Hospitals.
E	Hazardous Storage & Factories	1	Hazardous and flammable storage.
		2	Less hazardous and flammable storage.
		3	Woodworking, shops, factories; loose combustible fibers or dust.
		4	Repair garages.
		5	Aircraft repair hangars.
F	General Commerical	1a	Regular gasoline & service stations, non-vital vehicle storage garages.
		1b	Storage and parking of emergency vehicles (ambulances, utility trucks, etc.).
		2a	Wholesale stores, general warehouses.
		2b	Retail stores, includes drinking and dining under 100.
		2c	Office buildings, low rise, up to 75' height.
		2d	Office buildings, high rise, over 75' height.
		2e	Printing shops, Factories, Industrial Plants.
		2f	Police & Fire Stations, Communication Centers.

C8 Cont.

(TABLE C8-3 CONT.)

Group Letter	General Classification	Sub-Group Code No.	Occupancy Description
		2g	Warehouse, emergency supplies storage (medical, foodstuffs, chemicals, etc.).
		3	Aircraft hangars, Open parking garages.
G	Special Facilities (includes existing low fire hazard)	1	Ice plants, factories & workshops using non-combustibles, non-explosives
		2	Life line facilities, Utilities, Power Plants.
H	Hotels & Apartment Houses	1	Hotels, convents, monasteries.
		2	Apartment houses, low rise, up to 75' in height.
		3	Apartment houses, high rise, over 75' in height.
I	Dwellings		Dwellings and lodging houses.
			and sheds.
		2	Fences over 6' height, tanks and towers.

SOURCE: Using Uniform Building Code, 1973 Edition, as a point of departure, modifications and additions were made to Occupancy Group Types.

TABLE C8-4

FINAL GROUPING OF OCCUPANCIES

Group III: Buildings housing critical facilities which are necessary to post-disaster recovery and require continuous operation during and after an earthquake. The term critical facilities and emergency is defined as meaning designated by the governmental entity having jurisdiction.

Examples:

Fire facilities
Police facilities
Hospital facilities with emergency treatment facilities
Emergency preparedness centers
Emergency communications center
Power stations and other utilities required as emergency facilities

Group II: Buildings housing dense occupancies having a high transient population and/or sleeping conditions, or critical facilities requiring operation in the immediate post-disaster period. Restricted movement facilities.

Examples:

Public assembly for 100 or more persons
Open air stands for 2,000 or more persons
Day care
Schools
Colleges
Retail stores $\geq$ 5,000 square feet floor area per floor or more than 35 feet in height
Shopping centers with covered malls over 20,000 square feet gross area excluding parking
Office over four stories in height or more than 10,000 square feet per floor
Hotels over four stories in height
Apartments over four stories in height
Emergency vehicle garages
Detention facilities
Ambulatory health facilities
Hospital facilities other than those in Group III
Wholesale stores over four stories in height
Factories over four stories in height
Printing Plants over four stories in height
Hazardous occupancies consisting of flammable or toxic gases, flammable or toxic liquids including storage facilities for same

(TABLE C8-4 CONT.)

Group I: Low density occupancies and generally low transient population.

Examples:

Aircraft hangars
Woodworking facilities
Factories four stories or less
Repair garages
Service stations
Storage garages
Wholesale stores four stories or less
Printing plants four stories or less
Ice plants
Dwellings, single and two family
Townhouses
Retail stores less than 5,000 square feet per floor and 35 feet or less in height
Public Assembly for less than 100 persons
Offices four stories or less in height or less than 10,000 square feet per floor
Hotels four stories or less in height
Apartment houses four stories or less in height

Multiple-Occupancy Structures

At some time in the future, judging from recent architectural trends, mega-structure type buildings with multiple-occupancy groups will be designed or constructed. Due to economic pressures on the cost of construction, cost of travel and high values of land, Shopping, Living, Entertainment, Medical and Working Facilities may be combined and designed into a single structure. Any "preconceived boxes", or occupancy classifications within which buildings are classified must be designed to take into consideration the possibility of multiple-occupancy type structures. Some of the new Convention Centers, or Regional Shopping Center Malls, are in this category and represent a high-occupancy risk situation.

In this case it was concluded that the architectural systems and components are even more critical than in conventional type buildings. Egress and accessibility to these structures are most important.

TENTATIVE MATRIX

	COMPONENTS	A. ASSEMBLY 1 Public Assembly 100 or More	B. SPEC. ASSEMBLY 1 Open Air Stands	B. 2 Shopping Centers	C. EDUCATION 1 K Thru 12 51 or More	C. 2 K Thru 12 50 or Less	C. 3 Day Care	C. 4 College 51 or More	C. 5 College 50 or Less	D. CONFINED FACILITIES 1 Restrainted	D. 2 Nurseries Non-Ambulatory	D. 3 Ambulatory	D. 4 Hospitals	E. HAZARDOUS 1 Hazardous & Flam. Storage	E. 2 Less Hazard & Flam. Stor.	E. 3 Wood Working Factories	E. 4 Repair Garages	E. 5 Aircraft
1.1	BUILDING ACCESSIBILITY	S	S	G	S	G	↔	↔	G	G	G	G	S	G	G	G	L	L
2.1	EXTERIOR NON-STRUCTURAL WALLS	G2	G2	G2	G1	↔	↔	↔	G1	G	G	G	S	L4	↔	↔	↔	L4
2.2	VENEERS, SMALL SCALE	G1	G1	G1	G1	↔	↔	↔	G1	G			G	L	↔	↔	↔	L
2.3	CANOPIES	G1	G1	G1	G1	↔	↔	↔	G1	L	L	L	G	L	↔	↔	↔	L
2.4	ROOFING UNITS	G2	G	G2	G1	↔	↔	↔	G1	L	L	L	G	N	↔	↔	↔	N
2.5	CONTAINERIZED & MISC. ELEMENTS	G	G	L	G	L	G	L	L	L	L	L	G	N	↔	↔	↔	N
3.1	FIRE DETECTION	G	G	S	S	G	S	G	G	S	↔	↔	S	G	↔	↔	↔	G
3.2	FIRE SUPPRESSION	S	S	S	S	↔	↔	↔	S	S	↔	↔	S	S	↔	↔	↔	S
3.3	LIFE SAFETY COMMUNICATION	S	S	G	S	S	S	G	G	S	S	G	S	S	S	S	G	G
3.4	SMOKE REMOVAL	S	L	G	S	↔	↔	↔	S	S	↔	↔	S	G	S	G	L	L
4.1	STAIRS	S	S	S	G3	↔	↔	↔	G3	G2	G2	G2	S	G2	↔	↔	G3	G
4.2	ELEVATORS	L6	L	N6	L2	↔	↔	↔	L6	G2	G2	G2	S	L5	↔	↔	↔	L5
4.3	VERTICAL SHAFTS	L2	L2	L3	L3	↔	↔	↔	L2	L3	L3	L3	S	G2	L3	↔	↔	L3
5.1	HORIZONTAL EXITS	S	S	G	S	G	S	G	G	G	G	G	S	S	G	↔	↔	G
5.2	PUBLIC CORRIDORS	G	G	G	G	↔	↔	↔	G	G	G	G	S	G	L	↔	↔	L
5.3	PRIVATE CORRIDORS	L	L	L	G	G	G	L	L	G	G	L	S	L	N	↔	↔	N
6.1	FULL HEIGHT AREA & SEPARATION PARTITIONS	G	G	G	G	↔	↔	↔	G	S	G	G	S	S	4	↔	↔	G
6.2	FULL HEIGHT STRUCTURAL FIREPROOFING	S	S	S	G	↔	↔	↔	S	G	G	G	S	S	↔	↔	↔	S
6.3	FULL HEIGHT OTHER PARTITIONS	L	L	G	G	L	G	L	L	G	↔	↔	G	L	↔	↔	↔	L
6.4	PARTIAL HEIGHT PARTITIONS	L	L	L	G	L	G	L	L	L	G	L	G	N	↔	↔	↔	N
7.1	CEILINGS - FIRE MEMBRANE	S	G	G	G3	↔	↔	↔	G3	G	G	G	S	S	↔	↔	↔	S
7.2	CEILINGS - NON-FIRE MEMBRANE	S	L	G	G	L4	G	L4	L4	L4	L4	L4	G	L	↔	↔	↔	L
8.1	EQUIPMENT - CEILING MOUNTED	G	G	G	G	↔	↔	↔	G	S	G	G	S	L	↔	↔	↔	L
8.2	EQUIPMENT - WALL MOUNTED	G	S	L	S	L	G	L	L	G	G	G	S	L	↔	↔	↔	L
8.3	EQUIPMENT - FREE STANDING, UNSTABLE	G	G	G	G	↔	↔	↔	G	S	G	G	S	G	L	↔	↔	L
8.4	EQUIPMENT - FREE STANDING, STABLE	L	L	N	L	N	L	N	N	L	L	L	S	L	N	↔	↔	N
9.1	FURNITURE - UNSTABLE	G	G	L	L	N	L	N	N	L	G	L	G	N	↔	↔	↔	N
9.2	FURNITURE - STABLE	L	L	N	N	↔	↔	↔	N	N	N	N	L	N	↔	↔	↔	N
10.1	ART WORK - CEILING MOUNTED	G	S	G	G	↔	↔	↔	G	G	↔	↔	G	L	↔	↔	↔	L
10.2	ART WORK - WALL MOUNTED	G	S	L	G	L	G	L	L	L	↔	↔	L	L	↔	↔	↔	L
10.3	ART WORK - FREE STANDING, UNSTABLE	G	S	G	G	↔	↔	↔	G	L	G	L	S	L	N	↔	↔	N
10.4	ART WORK - FREE STANDING, STABLE	L	L	N	L	N	L	N	N	N	N	N	L	N	↔	↔	↔	N

	COMPONENTS	F. GENERAL COMMERCIAL 1a Serv. Station Stor. Garage	F. 1b Emergency Vehicles	F. 2a Wholesale Gen. Warehouse	F. 2b Retail	F. 2c Offices Low Rise	F. 2d Offices High Rise	F. 2e Paint Plants Factories	F. 2f Police Fire	F. 2g Warehouse Med Food	F. 3 Hangers	G. SPECIAL 1 Ice Plants	G. 2 Power Utilities	H. HOTEL, APTS 1 Hotels	H. 2 Apartments Low Rise	H. 3 Apartments High Rise	I. DWELL. 1 Dwellings	J. MISC. 1 Private Garages	
1.1	BUILDING ACCESSIBILITY	L	S	G	G	G	S	G	S	S	L	L	G	G	G	G	L	L	1.1
2.1	EXTERIOR NON-STRUCTURAL WALLS	L5	G	L6	G	G	G	L3	S	G	L3	L5	L6	S	G1	S	L	N	2.1
2.2	VENEERS, SMALL SCALE	L6	G	L3	G	G	G	L3	S	G1	L	L	L	G	G1	G	L	N	2.2
2.3	CANOPIES	L2	G	L6	G	G	S	L6		G1	L6	L	L	G	G1	G	L	N	2.3
2.4	ROOFING UNITS	L1	L1	L	G	G	G	L	G	G	L	L	S	S	S	G	N	N	2.4
2.5	CONTAINERIZED & MISC. ELEMENTS	N	L	L	G	G	G	L	G	L	L	L	L	G	S	G	N	N	2.5
3.1	FIRE DETECTION	L	G	L	G	G	S	G	S	G	L	L	G	S	G	S	S	L	3.1
3.2	FIRE SUPPRESSION	S	S	G	S	↔	↔	↔	S	S	G	G	S	S	S	S	S	S	3.2
3.3	LIFE SAFETY COMMUNICATION	N	S	S	S	G	S	L	S	G	L	L	G	S	G	S	L	L	3.3
3.4	SMOKE REMOVAL	S	S	L	G	G	S	G	S	G	L	L	S	S	G	G	L	L	3.4
4.1	STAIRS	G	G2	G	G3	G	S	G	S	G	G	G	G	S	G	S	S	L	4.1
4.2	ELEVATORS	L	L6	L6	G	G6	G	G	S	G	L	L	G	S	G6	S	N	N	4.2
4.3	VERTICAL SHAFTS	L3	L3	L3	G	L3	S	L3	S	G	L3	L3	G	S3	S	S	N	N	4.3
5.1	HORIZONTAL EXITS	G	G	G	↔	↔	G	G	S	G	L	G	G	G	G	G	-	-	5.1
5.2	PUBLIC CORRIDORS	G	G	L	G	G	S	G	S	L	G	G	G	S	G	G	-	-	5.2
5.3	PRIVATE CORRIDORS	L	G	N	L	G	G	L	G	N	L	L	L	G	L	L	N	-	5.3
6.1	FULL HEIGHT AREA & SEPARATION PARTITIONS	L3	G	L	G	G	S	G	S	S	L3	L3	G	G	L	G1	G	L	6.1
6.2	FULL HEIGHT STRUCTURAL FIREPROOFING	L3	G	G	G	G	S	↔	↔	S	S	G	S	G2	S	S	L	L	6.2
6.3	FULL HEIGHT OTHER PARTITIONS	N	L	L	↔	↔	↔	L	G	N	L	L	G	S	L	L	L	N	6.3
6.4	PARTIAL HEIGHT PARTITIONS	N	L	L	↔	↔	↔	L	G	N	L	L	G	G	L	L	L	N	6.4
7.1	CEILINGS - FIRE MEMBRANE	L3	G	S	G	G	S	↔	↔	S	G	G	G	S	G	S	L	L	7.1
7.2	CEILINGS - NON-FIRE MEMBRANE	L	L	L	↔	↔	↔	L	G	L	L	L	G	G	L4	L4	N	N	7.2
8.1	EQUIPMENT - CEILING MOUNTED	L	G	L	G	↔	↔	G	S	S	L	L	S	G	S	G	L	N	8.1
8.2	EQUIPMENT - WALL MOUNTED	L	G	L	G	G	G	L	S	G	L	L	S	G	S	G	L	N	8.2
8.3	EQUIPMENT - FREE STANDING, UNSTABLE	L	S	L	G	↔	↔	G	S	G	N	L	S	G	G	G	N	N	8.3
8.4	EQUIPMENT - FREE STANDING, STABLE	N	L	N	L	L	L	L	G	L	N	L	S	L	L	L	N	N	8.4
9.1	FURNITURE - UNSTABLE	N	L	N	G	L	L	L	G	L	N	L	G	G	L	L	N	N	9.1
9.2	FURNITURE - STABLE	N	N	N	L	N	N	N	L	L	N	L	G	L	N	N	N	N	9.2
10.1	ART WORK - CEILING MOUNTED	L	L	L	G	↔	↔	↔	↔	G	L	L	S	G	L	G	L	N	10.1
10.2	ART WORK - WALL MOUNTED	L	L	L	G	G	G	L	G	G	L	L	G	S	L	G	L	N	10.2
10.3	ART WORK - FREE STANDING, UNSTABLE	L	L	L	G	G	G	L	G	G	L	L	G	G	L	G	N	N	10.3
10.4	ART WORK - FREE STANDING, STABLE	N	N	N	L	L	L	N	L	L	N	N	G	L	N	L	N	N	10.4

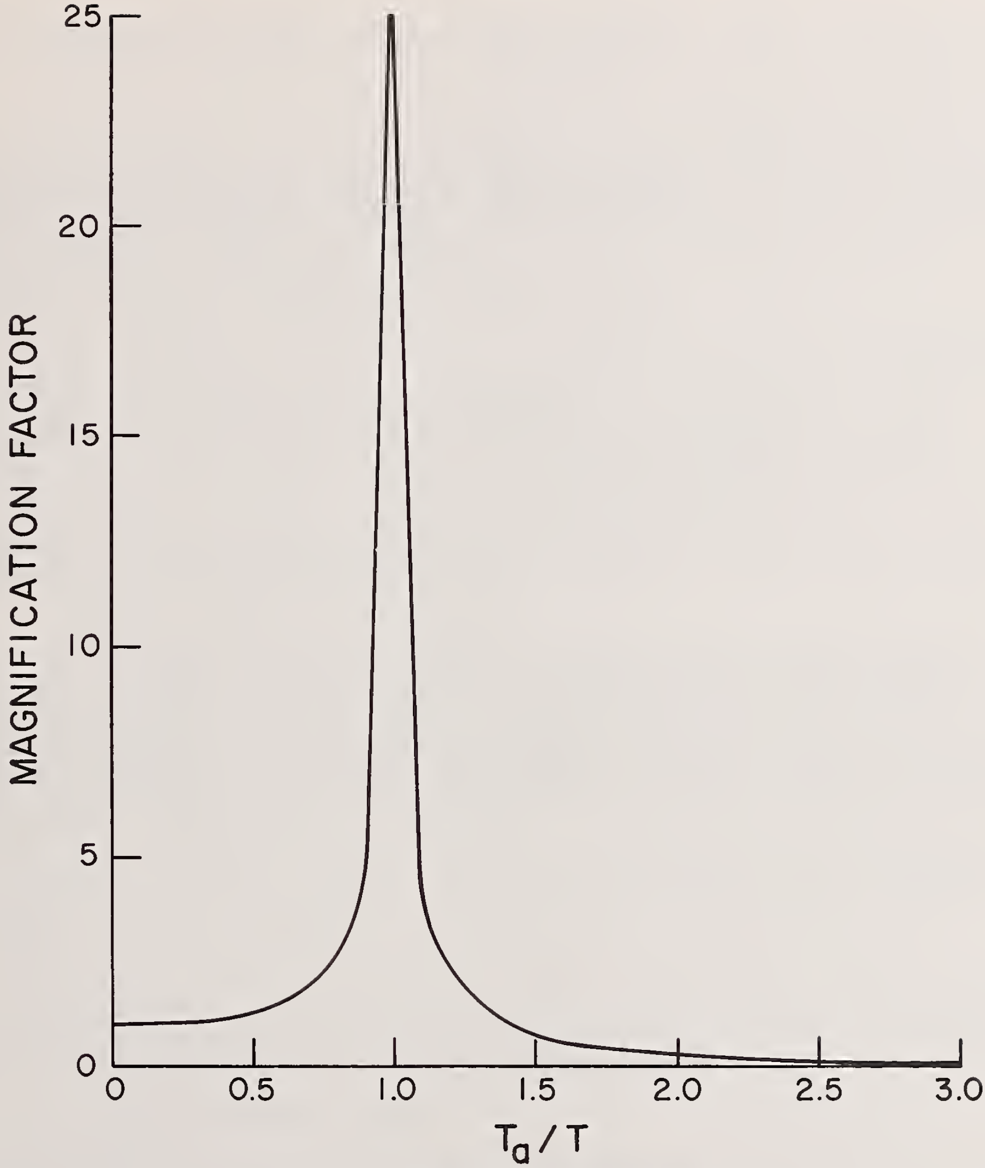
GENERAL NOTES:

1. OCCUPANCIES OCCUPYING A MINOR PORTION OF ANOTHER BUILDING SHALL NOT HAVE ANY COMPONENT CRITERIA OF A LOWER RATING THAN THE BASIC BUILDING.
2. WHERE ONE COMPONENT IS SUPPORTED BY ANOTHER COMPONENT, THE SUPPORTING COMPONENT MUST HAVE A PERFORMANCE LEVEL EQUAL TO OR GREATER THAN THE SUPPORTED COMPONENT.
3. WHERE THE COLLAPSE OF ONE COMPONENT CAN SERIOUSLY DAMAGE AN ADJACENT COMPONENT, THE COLLAPSED COMPONENT MUST HAVE A PERFORMANCE LEVEL EQUAL TO OR GREATER THAN THE ADJACENT COMPONENT.

PERFORMANCE NOTES:

1. MAY BE REDUCED ONE LEVEL, IF PROPERLY LANDSCAPED.
2. MAY BE REDUCED ONE LEVEL, IF PROPERLY LANDSCAPED & BUILDING IS ONLY ONE STORY.
3. MUST BE RAISED ONE LEVEL, IF BUILDING IS MORE THAN THREE STORIES OR 40 FEET HIGH.
4. MUST BE RAISED ONE LEVEL, IF NOT LIGHT WEIGHT; METAL FRAMES MUST STAY ATTACHED TO BUILDING.
5. MUST BE RAISED ONE LEVEL, IF BUILDING IS IN URBAN AREA, SYSTEM MUST STAY ATTACHED TO BUILDING.
6. ELEVATOR DOES NOT NEED TO OPERATE IF BUILDING IS LESS THAN 40 FEET HIGH.

TABLE C8-5



MAGNIFICATION FACTOR
VERSUS PERIOD RATIO

PLATE C8-1

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COMMENTARY

CHAPTER 9: WOOD

Sec. 9.1 REFERENCE DOCUMENTS

Unlike some structural materials such as concrete or steel, wood construction practices have not been codified in a form that is standard throughout the country. While heavy timber design practices generally follow the National Design Specifications for Stress Grade Lumber and its Fastenings (NDS), this document does not specify either simple or critical construction practices. There is a similarity of construction in lightweight wood framing throughout the country, but there is no single code of practice that is generally accepted. The closest approximation is probably Chapter 25 of the Uniform Building Code (UBC). Other reference documents are listed in Sec. 9.1.

It is not illogical to suggest that the framing practices specified in the UBC document be used throughout the country since wind design often governs over earthquake design even in highly seismic areas. The practices used for earthquake resistance are in large part those used to provide wind resistance.

The general provisions of Chapter 9 specify the construction requirements necessary to provide earthquake resistance, although many are also related to gravity load resistance. Since these requirements are not covered in any comparable document except the UBC, they are included here for clarity and completeness.

Sec. 9.2 STRENGTH OF MEMBERS AND CONNECTIONS

Since the loading provisions of Chapters 3 and 4 are based on a level of load resistance at yield point while normal code timber stresses must consider factors of safety, long-term deflection, etc., some adjustment must be made to tabulated stresses as given in the reference documents. This adjustment has been set at 200 percent of basic working stresses with the strength of members and connections subject to seismic forces acting alone or in combination with other prescribed loads being determined using the appropriate capacity reduction factors given in Sec. 9.2.

In the case of steel, the corresponding point has been averaged at about 1.7 times the tabulated working stress limitations. In the case of concrete, the adjustment is about 1.4. Capacity reduction factors are also specified for steel and concrete.

Wood has a variety of load factors and many of the accepted stresses do not have a constant relationship to an elastic limit or even an ultimate limit. When determining the factor for wood, consideration was given to the time effect of loading, the normal variability in strengths as related to both wood density and defects, and manufacture.

Sec. 9.3 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A are required to meet minimum construction as required without consideration of seismic forces except for anchorage of walls to floors and roofs as specified in Sec. 3.7.6.

Compared to present practice in many parts of the United States where recent editions of the UBC are not used, minimum wall bracing is required for wood frame buildings three stories in height to prevent racking. These are similar to the FHA minimum construction requirements. One common form of bracing has been omitted: let-in 1 x 4 or 1 x 6 diagonal bracing members. The original tests for this type of bracing were reported in "The Strength and Rigidity of Frame Walls", USDA Forest Products Laboratory, October, 1929; however, in those tests the let-in bracing was combined with horizontal timber sheathing boards. The San Fernando earthquake demonstrated that the expected strength is greatly reduced when sheathing boards are not used.

Sec. 9.4 SEISMIC PERFORMANCE CATEGORY B

Buildings assigned to Category B construction are required to meet requirements which are somewhat more restrictive than those for Category A. Materials (such as screws, lag screws, fiberboard diaphragms, and eccentric timber joints) and practices that have performed poorly in past earthquakes are regulated.

Sec. 9.5 SEISMIC PERFORMANCE CATEGORY C

The additional requirements for buildings assigned to Category C correspond roughly to the requirements for ordinary construction in highly seismic areas of the United States. Only timber or plywood diaphragms are permitted and the other related materials are limited for bracing purposes to the top floor of a timber building.

Sec. 9.6 SEISMIC PERFORMANCE CATEGORY D

The requirements for buildings assigned to Category D further restrict the use of plaster, gypsum, particle board, wallboard and fiberboard as bracing elements and require blocked diaphragms. These requirements apply only to those essential facilities in areas with the highest seismic exposure in the United States.

Sec. 9.7 CONVENTIONAL LIGHT TIMBER CONSTRUCTION

Conventional light timber framing consists of light framing where sizes of studs, joists, and rafters are generally determined from tables and construction details are based on common practice possibly modified by local building codes or FHA Minimum Property Standards. These buildings are often sheathed with non-timber materials such as plaster, sheet rock, particle board or other similar materials. Lateral resistance to wind or earthquake is usually not calculated but is determined by empirical rules such as are noted in Sec. 9.3.1 and 9.7.2.

Sec. 9.8 ENGINEERED TIMBER CONSTRUCTION

Engineered construction includes timber framed buildings where loads and forces are calculated and the required resistance is provided according to the tested or designed capacity of the resisting elements. Special requirements (including those for torsion) are given for all types of shear panel construction including diagonal sheathing, plywood, and other materials.

COMMENTARY

CHAPTER 10: STEEL

Sec. 10.1 REFERENCE DOCUMENTS

The reference documents are the current standard specifications for design of steel members and their connections in buildings as approved by the American Institute of Steel Construction, American Iron and Steel Institute, and the Steel Joist Institute. As future editions become available, suitable changes to the modifications in the succeeding sections should be made.

Sec. 10.2 STRENGTH OF MEMBERS AND CONNECTIONS

The modifications to standard specifications necessary to make them compatible with the design requirements and force levels specified in Chapters 4 and 5 and those made to minimize potential brittle modes of failure are specified. Capacity reduction factors are provided so that in the future explicit determination of member strength factors can be expedited. The modifications only affect designs involving seismic loads.

The capacity reduction factor of 0.9 for members and connections was selected primarily to account for uncertainties in design and construction. Connections of members have generally been a critical element in failures during past earthquakes. Therefore, a capacity reduction factor of 0.67 was introduced to increase the capacity of those connections which do not develop the full strength of the member. A ϕ factor of 0.8 was selected for partial penetration welds subjected to tension stresses because there has been little experience with this type of connection in past earthquakes.

It has frequently been found that optimum performance is obtained if connections fully develop the minimum capacity of the members of the seismic resisting system framing into a joint. Somewhat brittle-type failures have been observed when the capacity of connections are reached before that of the member. In order to provide a greater than usual margin of safety on braced frame connections, SEAOC (1974) provides that connections are to be sized without consideration of the one-third increase usually permitted unless the member capacity is fully developed. For moment frames, SEAOC (1974) requires that connections develop the member capacity. This concept is extended to moment frames by providing the same conservatism for moment frame connections as for braced frame systems.

It has been demonstrated by tests that a moment connection composed of welded flanges with a bolted web connection designed to carry the shear can develop the plastic capacity of steel sections. (Huang 1971, 1973; Regec 1972; Rentschler 1973-1976; Parfitt 1974; Popov 1970.)

When designing the connection to fully develop the member, the strengths of the connecting parts are determined using the factor in Sec. 10.2.1. This creates a step function in determining strengths. In design, however, a decision is made initially on whether or not the member strength will be developed so that the step should not create a design problem.

10.2.1 STRUCTURAL STEEL

Modifications are given for Reference 10.1 (AISC Specifications).

(A) The load effects determined from the load combinations specified in Sec. 3.7.1 are required to be equal to or less than the actual strengths of members and connections. The allowable stress levels specified in Part 1 of Ref. 10.1 do not identify this condition and are not applicable. It is assumed, unless specifically described otherwise, that the strengths are linear, elastic allowable stresses modified to meet the elastic limit of the structure. The design for the combination of dead and live loads and impact, if any, is not modified from the current specifications. Information leading to the determination of member and connection strengths is being studied but the results are not available for inclusion in these provisions. Future research may be able to better define member and connection strengths for resisting seismic load effects. These may be strengths related to a mean value or a given deviation from the mean. Also future development may indicate that varying ϕ factors would be appropriate for different types of members as indicated in current proposals (Galambos 1973-1976). A modifier of 1.7 and a capacity reduction factor of $\phi = 0.9$ on working stress values were chosen after a review of a number of items such as:

1. The margin of safety between the yield strength and allowable stress of short columns.

2. The margin of safety between the yield strength and allowable tensile stress.

3. The margin of safety of compression members varies between 1.7 and 1.9 (Ref. 10.1 and Johnston 1976).

4. The increase permitted on connecting devices in Part II of Ref. 10.1 is 1.7 (Ref. 10.1). The actual margin of safety is often higher (Fisher 1974, Galambos 1976 No. 33).

(B) The allowable shear stress specified in Sec. 1.5.1.2 of the AISC specifications is 0.40 F_y. When multiplied by 1.7, the value becomes 0.68 F_y. This is higher than the 0.55 F_y given in Sec. 2.5. This difference is discussed in the Commentary of the AISC specifications and the higher value was permitted because there had been no experience to modify it. When the shear stress in a member or joint results primarily from forces generated by earthquake motions, it is felt that the more conservative approach given in Part 2 of the AISC specifications should be used. It is anticipated that this requirement would apply primarily to unbraced frame members and joints. Future research may indicate that the shear limit for resisting seismic load effects should be modified.

(C) As the level of design is the same as contemplated in the definition of P_e on Page 5-60 of Ref. 10.1, the 12/23 modifier of F_e' is removed.

(D) Proportioning members of seismic resisting braced frame systems of a building which has been designed by plastic analysis for gravity loads shall be based on the strength of members as specified in Part 2 of Ref. 10.1. However, the analysis shall be based on the elastic analysis described in ATC-3 Sec. 3.1. Thus the current references to plastic analysis methods and the load factors are not used.

(E) This Subsection provides modification to the interaction equations when the P-delta effects are explicitly determined in conformance with Sec. 4.6.2. In columns, the reductions given to the allowable stresses are in part a result of the consideration of member P-delta effects. These P-delta reductions are modified in Ref. 10.1 by a K factor which is a recognition of the effect of end restraint in the member P-delta relationship. In beam-columns, the P-delta effect is also considered as an increase (or decrease) to the moments at the end of the columns expressed as a function of:

$$C_m/1 = \frac{f_a}{F_e'}$$

(Ref. 10.1, Johnson 1976, Galambos 1968). The bases for the values of this ratio in braced systems are well documented. The selection of the value of C_m in unbraced frames was an approximation applicable primarily to designs where significant applied horizontal forces are not present. Since the advent of computer analyses, the solution of the secondary effects resulting from deflection has become much easier. In most cases, with significant horizontal force displacements (but limited by drift requirements) the first iteration of deflection is sufficient. It is possible that some members, such as weak axis columns depending on end support conditions, may have critical stress occur at the mid-story rather than the column ends. Thus the stress limits specified for braced frames should not be exceeded.

10.2.2 COLD FORMED STEEL

The allowable stress levels of Ref. 10.2 and 10.3 are not applicable to the force levels in the earthquake analysis specified in Chapter 3. As an interim measure the strengths of the members governed by these provisions are determined using basic stresses increased by 1.7 and using $\phi = 0.9$.

10.2.3 STEEL CABLES

The allowable stress levels of steel cable structures specified in Ref. 10.6 are modified for seismic load effects. The value of 1.5 T_4 was chosen as a reasonable value to compare with increases given to other working stress levels.

Sec. 10.3 SEISMIC PERFORMANCE CATEGORY A

No special requirements for seismic design of buildings assigned to Category A were deemed necessary.

Sec. 10.4 SEISMIC PERFORMANCE CATEGORY B

Detail requirements for buildings assigned to Category B are given.

10.4.1 ORDINARY MOMENT FRAMES

Where moment resistant frame systems are used for the seismic resisting system, they shall be Ordinary Moment Frames. Ordinary Moment Frames are assumed to respond to the design earthquake by requiring a limited amount of nonlinear behavior. For this type of moment frame, proportioning of members and their connections is based on the requirements of the referenced specifications as modified by Sec. 10.2 for making working stress values compatible with seismic design. For these types of frames no change is provided to local buckling criteria of Appendix C of Ref. 10.1 and in Ref. 10.2 and 10.3.

10.4.2 SPACE FRAMES

Space frames when used shall conform to Ref. 10.1 or 10.2 or 10.3.

Sec. 10.5 SEISMIC PERFORMANCE CATEGORIES C AND D

The requirements for buildings assigned to Category C or D are given.

10.5.1 SPECIAL MOMENT FRAMES

Where a moment resisting frame system is used as the seismic resisting system it shall be a Special Moment Frame as specified in Sec. 10.6. An exception is permitted for one- and two-story buildings assigned to Category C; Ordinary Moment Frames may be used. This exception is based on the generally good experience record of such buildings during earthquakes.

Minor structures and structures with light metal or wood cladding designed without special requirements for nonlinear ductile behavior have performed well even during strong earthquakes. However, major structures in areas of high seismicity and those minor structures housing emergency occupancies should be provided with the full provisions for inelastic performance specified by Sec. 10.6. A major structure in this instance is defined as a building over two stories. It is conceivable that some one- and two-story structures should be considered major structures and that some buildings of four or five stories, particularly those with light flexible cladding, should not be classified as major structures. Some judgment and leniency should be exercised in enforcing the two-story limitation.

10.5.2 BRACED FRAMES

Braced frames are designed to either carry both tension and compression or to carry tension only, such as rod or strap bracing. There are insufficient data on the nonlinear behavior of braced systems with which to develop definitive guidelines for adequate performance. Braced systems have performed well when adequately designed and detailed. Designs using the tension-only concept have resulted in a rather large amount of damage to adjoining elements. Therefore, until detailing requirements for providing adequate nonlinear behavior in braced systems are determined, it is recommended that in high seismic areas the tension-only concept not be used for major structures. As discussed in the previous paragraph, leniency should be exercised in enforcing the two-story limitation.

Sec. 10.6 SPECIAL MOMENT FRAME REQUIREMENTS

Structures having Special Moment Frames designed to meet the requirements of Sec. 10.6 are intended to have the capability of significant nonlinear deformation. The sizing of members is based on the limit of an elastic model as specified in ATC-3 Sec. 3.1. The nonlinear capability is provided by meeting the special requirements in this subsection.

1. The statement regarding M_p is added to the specifications so that it can be used to define the flexural strength of a frame member. This definition of strength is obviously not the elastic limit of the member but as a consequence of strain hardening it is felt to be a reasonable limit to represent the point at which the frame as a whole will start to substantially deviate from linear response. The fact that the mean yield strength of the material is in excess of the minimum specified yield strength also supports this design concept.

2. For this type of moment frame the steels to be used are limited to those whose properties are similar to the steels used in tests to demonstrate the nonlinear behavior of structural members and joints. (Lehigh 1967-1976, Popov 1970, 1975; Bertero 1973; Krawinkler 1971; Becker 1971.) Other steels exhibiting similar ductility and strain hardening characteristics such as those listed would also be appropriate.

3. Sec. 2.3.1 of Ref. 10.1 is deleted as not applicable to unbraced frames. The maximum axial load on columns of 0.6 P_y for Special Moment Frames is provided to reflect the recommendations from recent tests. The upper limit for the axial forces is lowered from 0.75 P_y, as specified in Sec. 2.3.2 of Supplement No. 3 of Ref. 10.1, to 0.6 P_y because:

a. The uncertainties involved in predicting the maximum axial forces that can be induced during a severe earthquake are so great that it is convenient to be more conservative than in case of design for standard loadings.

b. Columns in a moment resisting frame system (ductile or nonductile) excited by severe earthquake ground motion can be subjected to cycles of inelastic moment reversals. Test results (Popov 1975) have shown that when a column is under a constant axial force

$P \geq 0.6\,P_y$ and is subject to reversals of moments inducing yielding, local buckling develops in the columns during first reversal of inelastic moment and when this occurs the axial force cannot be maintained.

4. The actual location of points of inflection in columns when the frame is deforming nonlinearly is not known. Thus the shear and moment requirements at a column splice are difficult to accurately assess. The use of partial penetration welds for column splices produces a point that could result in a brittle-type frame failure if the level of stress is critical at any time during the response of the frame. In order to provide a conservative guide to the determination of when partial penetration welds can be used, the following criteria are provided by the provisions:

a. A conservative estimate of joint moment capacities is required assuming the yield of the critical sections at the joint are 125 percent of the minimum specified yield strength.

b. The potential movement of the point of inflection within the column height is determined by assuming that one column joint is stressed to one-half of its plastic capacity and the other joint is stressed to its full plastic capacity.

c. The effect of vertical acceleration is considered by using the load combinations of Sec. 3.7.1.

In some cases columns do not have a point of inflection within a story height. For these cases it could be unconservative to design the splice to comply only with cases a and b above. Thus it is emphasized that the load effects resulting from the loads specified in Sec. 3.7.1 should also be considered.

5. In addition to the shear stresses resulting from the elastic analysis of the system under the specified loads, shear stresses should be determined based on the assumption that the full flexural strengths of the elements are reached through nonlinear displacement of the frame members. The critical sections may be either in beams or in columns. Frequently this may be only a nominal change in the shear design requirements. It is felt that the shear requirements should be consistent with the actual response of the frame to the design earthquake. If the members are oversized, the actual inelastic displacement of the frame will not be the same as assumed when assigning the load modifiers in Sec. 3.7.1. The resulting increase in the design shear can be significant.

Recent research has been performed on beam-column joint panel zones and methods have been proposed for determining the panel zone shear capacity with and without shear reinforcement. (Becker 1971, Bertero 1973, Krawinkler 1971.) Frequently panel zone shears have been determined assuming the joint moments equal to the sum of the beam (or columns) moment capacities on each side of the joint. This is a simple and conservative method of determining panel zone shears, but usually results in excessive reinforcement requirements. However, it is usually not possible to develop this joint moment on the frame before total frame instability occurs. Also formation of hinging by shear in restricted areas may provide stable nonlinear response. In most cases the provisions of Sec. 10.6 permit reduction in the amount of reinforcement required when an approximate frame analysis is made with deflections twice those determined using the prescribed forces. The factor of 2 is arbitrary but would provide elastic panel zone response well beyond the deformations represented by the design forces at the elastic limit of the structure.

Ref. 10.1 is not explicitly based on the condition of high shears plus bearing that is found in the joint of a moment resisting frame. In order to provide for this condition until further research is performed to provide a more workable equation, the buckling compressive stresses are combined parabolically with the shear stresses in the formula proposed by Newlin and Chen (Chen 1971). The development of this formula is as follows:

The Chen-Newlin formula is

$$t < \left[d_c^2 \sqrt{F_y} + 180\ c_1\ A_f\right] \frac{1}{125\ d_c\ \sqrt[4]{F_y}}$$

which was derived from a formula established in the report for the critical crippling stress on a column web, i.e.,

$$P_{cr} = \left(\frac{1.7\ \sqrt[4]{F_y}}{\sqrt[4]{36}} - \frac{d_c\ \sqrt[4]{F_y}}{180t}\right) d_c\ t\ F_y$$

This becomes

$$P_{cr} = \left(0.694\ \sqrt[4]{F_y} - \frac{d_c\ \sqrt{F_y}}{180t}\right) d_c\ t\ F_y$$

The concentrated load to be compared to this critical buckling load is $P = A_f\ (F_y)_F$ where A_f is the area of the beam flange and $(F_y)_F$ is the yield of the beam flange steel. This formula can be modified to $P = A_f C_1\ F_y$, where C_1 is the ratio of beam yield stress to column yield stress and F_y is the column yield stress.

If the effects of crippling and shear are combined in a second order interaction, the equation would be

$$\left(\frac{P'_{cr}}{P_{cr}}\right)^2 + \left(\frac{f_v}{0.55\ F_y}\right)^2 = 1$$

in which P'_{cr} is the critical buckling load when acting in conjunction with shear and P_{cr} is the critical buckling load given by the above equation acting without shear.

Substitutions of the above formulas yields the limiting relationship of

$$\left[\frac{A_f C_1\ F_y}{\left(0.694\ \sqrt[4]{F_y} - \frac{d_c\ \sqrt{F_y}}{180t}\right) d_c t F_y}\right]^2 + \left(\frac{f_v}{0.55\ F_y}\right)^2 = 1$$

Solving for t provides the formula

$$t < \left[d_c^2\ \sqrt{F_y} + \frac{180 C_1\ A_f}{1 - \left(\frac{f_v}{0.55\ F_y}\right)^2}\right] \frac{1}{125\ d_c\ \sqrt[4]{F_y}}$$

which is the formula in the provisions.

6. Connections usually should be designed to develop the joint capacity rather than the connection stresses resulting from the effects of the specified earthquake

loading. This is to ensure that ductile behavior will occur in the members. Connections could be devised, however, to be capable of providing adequate nonlinear response in themselves. This should be demonstrated by proper analyses or tests.

7. Sec. 2.9 of Ref. 10.1 is modified to delete reference to plastic design procedures for design of the seismic resisting system so as to be in conformance with the requirements for an elastic analysis as specified in Sec. 3.1.

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COMMENTARY

CHAPTER 11: REINFORCED CONCRETE

The main concern of Chapter 11 is the proper detailing of reinforced concrete construction for earthquake resistance. Calculations to determine the strength of structural components and members are to be based on Ref. 11.1 except for the modification or introduction of several strength reduction factors (Sec. 11.2), and expression for shear strength of walls (Equation 11-5) and allowable shear stresses in joints (Sec. 11.7.3). The exceptions refer to buildings assigned to Seismic Performance Categories C and D.

The locations of the main requirements of Chapter 11 for frames and walls resisting seismic forces are given in the following table.

	Category A	Category B	Categories C & D
Frame	ACI 318-71	Sec. 11.6	Sec. 11.7
Wall	ACI 318-71	ACI 318-71	Sec. 11.8

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 the requirements of Sec. 11.6 and ACI 318-71 (as referred to in Sec. 11.1) as well as those of Sec. 11.7.

Buildings in Categories C and D are required to have special details which are currently considered to be the minimum requirement for producing a monolithic reinforced concrete structure with adequate proportions and details to make it possible for the structure to sustain a series of oscillations into the nonlinear range of response without critical decay in overall lateral strength and without loss of basic structural integrity. The demand for integrity of the structure in the nonlinear range of response is consistent with the rationalization of design forces specified in Chapter 3.

The experience, both in the field and 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 Catgories C and D construction.

Sec. 11.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength reduction factors listed in Sec. 11.2 are intended to be used to modify the expressions defining section or element strength by ACI 318-71.

The factor ϕ = 0.5 listed for precast components reflects primarily the questions related to the projection of experience with monolithic structures to a form of construction differing fundamentally in fabrication.

The strength reduction factor, ϕ, of 0.5 is assigned to "tied columns" as a result of their observed poor performance in resisting seismic forces. The intent of this requirement is to discourage the use of columns in earthquake zones without special transverse reinforcement.

The strength reduction factor of 0.6 (for strength of components of which strength is governed by shear) is intended for brittle elements with very low span-to-depth ratios such as low-rise walls or portions of walls between openings. Proportions of those members may be such that it becomes impractical to reinforce them to have a nominal shear capacity exceeding that in flexure. In those cases, the shear capacity of the system must be assessed with the lower strength reduction factor of 0.6. This requirement does not apply to the proportioning of joints.

A dearth of laboratory and field experience related to the shear strength of structural components made with lightweight-aggregate concrete and subjected to load reversals is the reason for the reduction of values of ϕ modifying the shear strength of such components and their construction joints.

The allowable loads on anchor bolts have been chosen to suit the capacity reduction factors assumed in this document.

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. Such frames must have attributes of Ordinary Moment Frames as specified in Sec. 11.6. Structural (shear) walls of buildings in Category B are to be built in accordance with the requirements of ACI 318-71.

Sec. 11.5 SEISMIC PERFORMANCE CATEGORIES C AND D

11.5.1 MATERIAL REQUIREMENTS

A limit is set for the compressive strength of lightweight-aggregate concrete used in structural components of Special Moment Frames and walls proportioned to resist seismic forces because of lack of data on the behavior of such components under load reversals into the nonlinear range of response.

The use of longitudinal reinforcement with strength substantially higher than assumed for design may lead to shear or bond failures which are to be avoided even though such failures may occur at higher loads than those anticipated in the design. Therefore, an upper bound is set to the actual yield stress of the steel in relation to the specified yield stress.

To develop inelastic rotation at joints without developing critically large strains in bonded reinforcement, it is necessary to use steel with tensile strength well in excess of its yield stress. Consequently, the ratio of the actual tensile strength to the actual yield stress is required to be at least 1.25.

The specifications for ASTM A-706 have been developed to satisfy the basic requirements for earthquake resistance.

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

11.5.4 SUPPORT FOR DISCONTINUOUS COMPONENTS

Dynamic response analyses and field observations have indicated that columns supporting discontinued stiff structural elements, such as walls or vertical trusses, are subjected to repeated large axial loads and large lateral displacement reversals when the building is subjected to strong ground motions. Therefore, it is required that these columns have special transverse reinforcement throughout their length, not only to provide confinement near the joints but also to provide a uniformly strong and tough column.

Sec. 11.6 REQUIREMENTS FOR ORDINARY MOMENT FRAMES ASSIGNED TO CATEGORY B

The Ordinary Moment Frame is to be designed with certain special detail requirements to maintain its integrity under lateral displacement reversals.

11.6.1 FLEXURAL MEMBERS

The amount of longitudinal reinforcement in a flexural member of an ordinary frame is governed by three requirements.

1. The lower bound on the reinforcement ratio ($200/f_y$), refers to the concern about reinforcing beams so lightly that the flexural capacity is exceeded by the cracking moment of a section leading to the abrupt development of large rotation at that section when the section cracks.

2. The upper bound on the reinforcement ratio is a flat limit rather than the familiar form used by ACI 318-71 which refers to "balanced" failure conditions and sets the limit on the reinforcement ratio as a function of the yield stress of the reinforcement and the compressive strength of the concrete. For a section subjected to bending only and loaded monotonically to yielding, the option of expressing the amount of permissible tensile steel as a function of nominal limiting strain in the concrete, the amount of compression reinforcement, and the quality of the concrete is feasible because the likelihood of compressive failure can be estimated reasonably well with the behavioral model assumed for determining the amount of reinforcement it takes to produce a balanced failure corresponding to "simultaneous" yielding of steel and crushing of concrete. The same behavioral model, because of invalid assumptions (such as linear strain distribution, well defined yield point for the steel, constant limiting compressive strain in concrete, and intact shell concrete), fails to describe conditions in a frame member subjected to displacement reversals well into the nonlinear range. Thus, there is no rationale for continuing to refer to balanced conditions for limiting reinforcement ratio in earthquake resistant design of reinforced concrete structures. The flat limit of 0.025 is based primarily on considerations of steel congestion and indirectly on limiting shear stresses in girders of typical proportions.

3. The minimum limit of two No. 5 bars, top and bottom, refers again to construction rather than behavioral requirements.

The requirements for the ratios of negative-to-positive moment strengths along the length of flexural members are, in prismatic elements, to force the regions of inelastic rotation to develop near the faces of the connections where special transverse reinforcement is to be provided.

A bar to be terminated within a column must be extended through the column even if the anchorage length, computed from the face of the column, does not necessitate such a long extension. A standard 90-degree hook may be used to develop additional anchorage if a straight extension of the bar through the column does not suffice to anchor the bar.

Even though the design force combinations may indicate negligible shear at mid-span, induced by earthquake motions, the beams may develop substantial shear forces as a function of the amount of longitudinal reinforcement and depth-to-span ratio. Therefore, a minimum amount of web reinforcement is specified throughout the span. The stirrups are required to have at least two legs in order to develop a core of concrete having a finite thickness available to resist shear. The spacing of d/2 increases the likelihood of having at least one stirrup across any inclined crack. Deformed bars are required for their efficiency in restraining cracks. Formula 11-1 represents a crude but practical method of specifying a minimum quantity of reinforcement in those portions of the beam where shear may be critical as a result of local yielding under loading reversals. With other variables maintained constant, an increase in the amount of top or bottom longitudinal reinforcement increases the maximum possible shear force in the beam. Therefore, the amount of minimum web reinforcement is increased as a function of the longitudinal reinforcement.

The maximum spacing of stirrups is decreased to d/4 in regions of possible yielding (near ends of flexural components) to develop better and more uniform restraint for inclined cracks.

11.6.2 MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

This Subsection refers to details providing confinement for concrete and compression reinforcement in regions of possible yielding. If the columns are nonprismatic and/or have bars cut off at different levels, the designer is cautioned to make certain that transverse reinforcement, as specified, is provided in all regions where yielding is anticipated, a problem most practically solved by calling for transverse reinforcement throughout the length of the column.

Sec. 11.7 SPECIAL MOMENT FRAME REQUIREMENTS

Experimental studies of reinforced concrete elements subjected to cyclic loading (large displacement reversals) 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 subjected to monotonically increasing load; the necessary increase of web reinforcement to insure flexural failure being higher in the case of no axial load. This observation is reflected in the elimination of the term representing the contribution of concrete to shear resistance. Even if the columns in a frame may be designed to have a very low probability of yielding in the event of the anticipated earthquake, the consequences of a shear failure in a column are so grave that they must be provided with sufficient reinforcement to reduce the risk of shear failure in case they are loaded into the nonlinear range of response. It must also be emphatically pointed out that the elimination of the contribution of the concrete shear resistance from the design equation is simply a matter of convenience for design calculations. It should not be interpreted to mean that concrete is not required to resist shear.

11.7.1 FLEXURAL MEMBERS

The requirements of Sec. 11.7.1 are intended to apply to horizontal elements of frames such as beams or girders resisting lateral forces induced by earthquake motions. However, vertical components of frames which are subjected to a total axial compressive force of less than 0.1 $f_c'A_g$ and which are within the geometric limitations listed under 11.7.1 may be designed in accordance with the requirements of this section. On the other hand, if any horizontal component is subjected to an axial design compressive force exceeding 0.1 $f_c'A_g$, it must be proportioned and detailed in accordance with the requirements listed under 11.7.2 or, if it does not satisfy the requirements of that section, as a structural (shear) wall.

Experimental evidence indicates that under reversals of displacement into the nonlinear range, behavior of frame components having length-to-depth ratios of less than approximately four is significantly different from the overall behavior of relatively slender frame components. Design rules derived from experience with relatively slender frame components do not apply directly to components with length-to-depth ratios less than four, especially with respect to the shear strength of such stubby components under load reversals. Therefore, special frame components with length-to-depth ratios of less than four are to be proportioned and detailed as truss or wall components.

The geometric constraints given for flexural members are based primarily on past practice. The width-to-depth ratio is set to encourage compact cross-sections having a reasonably low risk of lateral instability in the nonlinear range of response. The maximum width limitation is related to the problem of efficient transfer of moment from girder to column. This 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 nonlinear range of response should be evaluated realistically considering the effect of cracking and reinforcement slip (at sections where bars have been terminated) rather than on the basis of gross section.

As indicated in 11.7.1(A), lap splices are prohibited at known regions of flexural yielding because they are unreliable under loading reversals. Transverse reinforcement for lap splices at any location is mandatory because of the likelihood of loss of shell concrete.

The requirement about the stagger of welded or mechanical splices is related to construction convenience and serviceability rather than to strength.

Transverse reinforcement is specified in 11.7.1(B) for two different requirements: (1) shear strength of the component and (2) confinement of the concrete and reinforcement in zones where there is a possibility of yielding under earthquake effects. For the first requirement transverse reinforcement is calculated using the relevant prescriptions of ACI 318-71 (Ref. 11.1) with the changes in capacity reduction factors (ϕ) and other requirements (e.g., $v_c = o$ in certain cases) in Chapter 11. Reinforcement for the second requirement is nominal as specified in 11.7.1(B). The amounts of transverse reinforcement required for shear and confinement are not additive.

The total shear distribution on a flexural component shall be determined from a free body of the element between two possible plastic hinges. An example is given in Fig. C11-1. Ultimate moments at the plastic-hinge locations are to be based on a steel stress equal to 1.25 f_y because of the likelihood of strain hardening at hinge locations as well as the possibility of having an actual yield stress in excess of the design yield stress.

Requirements listed for transverse reinforcement in 11.7.1(B) refer to considerations of confining the concrete and providing lateral support for the compression reinforcing bars in regions where yielding is anticipated.

In the case of flexural components where the strength of the component changes along the span or where the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span. In such cases, it is preferable to provide transverse reinforcement as specified throughout the length of the member rather than attempt to pinpoint the zones of yielding by calculation.

11.7.2 MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

The geometric constraints listed for columns of Special Moment Frames follow from previous practice.

The intent of 11.7.2(A) is to reduce the likelihood of flexural yielding in columns and to limit inelastic action to horizontal components which are typically subjected to small axial forces. Columns for which this requirement cannot be satisfied, such as columns supporting heavy transfer girders, should be treated as columns supporting discontinued stiff elements.

The lower limit on the column reinforcement ratio listed in 11.7.2(B) refers to the traditional concern for the effect of time-dependent deformations of the concrete as well as the desire to avoid a sizeable difference between flexural cracking and yielding moments. The upper bound to the reinforcement ratio reflects concern for steel congestion, load transfer from horizontal to vertical components in low-rise construction, and 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 columns in the event of strong ground motion, makes lap splices in those regions quite vulnerable. If lap splices are to be used at all, they must be located near the mid-height of the column where stress reversals are less likely to occur.

The main function for the transverse reinforcement specified in Sec. 11.7.2(C) is to provide confinement for the concrete and lateral support for the reinforcement. The amount of transverse reinforcement so required may also be used to resist shear.

Equation 10-3 of ACI 318-71 is based on the arbitrary concept that, under axial compressive loading, maximum capacity of the helically reinforced column (spiral column) before loss of shell concrete is equal to that with the shell concrete destroyed and the helical reinforcement stressed to its useful limit. The toughness of the "spiral column" under axial loading is not directly relevant to its typical role in earthquake-resistant structures where toughness or ductility is likely to be related to performance of the column under large reversals of moment as well as axial load. Nonetheless, without implicit quantitative relationships between performance criteria interpreted in terms of the quality of the confined concrete and the amount of spiral reinforcement, there has been no compelling reason to modify Equation 10-3 for earthquake-resistant construction other than adding Formula 11-2 which provides a varying lower bound to the amount of transverse reinforcement. Formula 11-2 tends to govern for columns with large cross-sectional areas.

A conservative evaluation of the available data from tests investigating the effect of rectilinear transverse reinforcement on the behavior of reinforced concrete columns would suggest that such reinforcement has little influence on strength under axial loading but improves ductility although not as effectively as spiral reinforcement. Consequently, it follows that if rectilinear transverse reinforcement is used to confine the concrete, there ought to be more of it per volume of concrete so as to have an effect comparable to that of spiral reinforcement. Formulas 11-3 and 11-4 compare respectively to Equations 10-3 and 11-1 of ACI 318-71. Formulas 11-3 and 11-4 require larger volumes of reinforcement per unit length of confined core of column.

Shear forces in columns are to be determined from the strength and geometric properties of the column itself as indicated in Fig. C11-2. The end moments on the free body of the column are based on the yield stress of the longitudinal reinforcement because of the low probability of having extensive inelastic response in the column. The intent of condition (2) in the last paragraph of 11.7.2 is to have the end moment equal to the maximum calculated moment that the column section may develop. If the characteristics of the column section are such that a larger moment may be developed at a lower compressive axial load than the maximum corresponding to the design conditions, the larger moment is to be used for determining the maximum shear.

11.7.3 JOINTS

Evaluations of existing and new data on the strength of joints subjected to moment reversals have indicated that the strength of the joint is relatively insensitive to the amount of transverse reinforcement, provided there is a minimum amount, and that a limiting shear stress of 16 $\sqrt{f_c'}$ for laterally confined and 12$\sqrt{f_c'}$ for unconfined joints may be used for normal weight-aggregate concrete. There are no directly relevant data on the strength of joints made with lightweight-aggregate concrete. The allowable stress for joints made with lightweight-aggregate concrete has been based on the observation that shear transfer in such concrete has been measured to be approximately 75 percent of that in normal weight-aggregate concrete.

Sec. 11.8 SHEAR WALLS, BRACED FRAMES, AND DIAPHRAGMS

Sec. 11.8 contains requirements for dimensioning and detailing of relatively stiff structural systems resisting lateral loads including parts of roof and floor systems transmitting inertia forces to the seismic resisting system. Special frame elements which constitute parts of the seismic resisting system and which do not qualify under Sec. 11.7 are also to be detailed under this section.

It is required that the vertical reinforcement ratio be equal to or in excess of the horizontal reinforcement ratio in order to avoid the possibility of having inadequate web reinforcement in walls which are short in comparison to their height. Splices are staggered in an effort to avoid weak sections.

The requirement for a minimum of two layers of reinforcement in walls carrying substantial design shears is based on the observation that, under typical construction conditions, the likelihood of maintaining the location of a single layer of reinforcement near the middle of the wall plane is quite low.

The calculated nominal stress of $0.2f_c'$ on a structural wall indicates that the integrity of the structure may depend on the ability of the material in that location to resist substantial compressive force under severe repeated loading. Therefore the boundary element with transverse reinforcement as specified in Sec. 11.8.4 is required in such locations to provide confinement for the concrete and compressive reinforcement.

Because the horizontal reinforcement in walls requiring boundary members is likely to resist shear force after development of inclined cracks, its anchorage in the boundary members is quite critical. The possibility, under very large earthquake forces, of tensile cracks in (and perpendicular to the axis of) boundary elements which may interfere with the anchorage of the horizontal reinforcement should be considered in developing anchorage details.

11.8.4 BOUNDARY MEMBERS

Determination of the size of the vertical boundary member at the edge of the wall is predicated on the assumption that the boundary member may have to carry all compressive forces at the critical section of the wall under the action of the maximum lateral forces caused by earthquake effects and gravity loads which the wall is designed to resist.

Diaphragm boundary members are to be proportioned to carry the forces necessary to develop the design moment plus any axial compressive force.

The intent of the requirement for boundary members around openings is to compensate for the strength reduction resulting from the opening. It is assumed that the designer will verify that the strength across an opening of a wall or diaphragm is adequate to resist the overall design forces.

11.8.5 BRACED FRAMES

Individual components of structural trusses or braced frames made of reinforced concrete are to be treated as components resisting compressive axial forces. However, truss components and their joint and splice details must be proportioned for design forces which may be tensile and compressive.

11.8.7 CONSTRUCTION JOINTS

This Section requires that construction joints be designed and constructed to resist seismic design forces at the joint. Formula 11-6 is based on Equation 11-30 of Ref. 11.1, but is restated to reflect dowel action and frictional resistance.

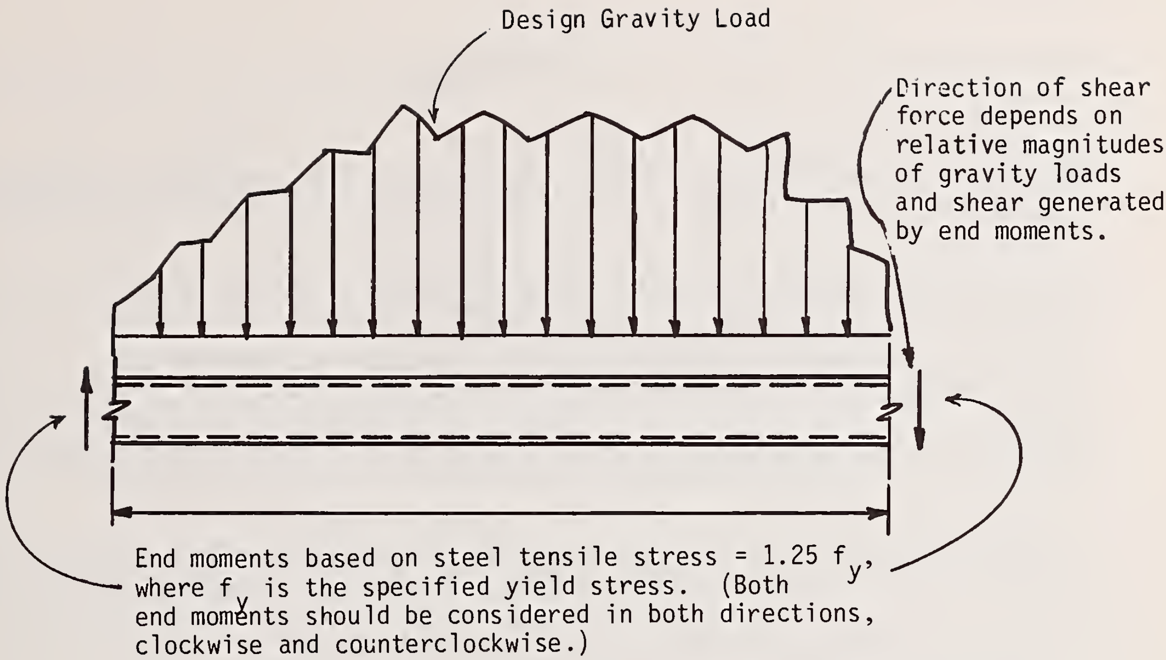
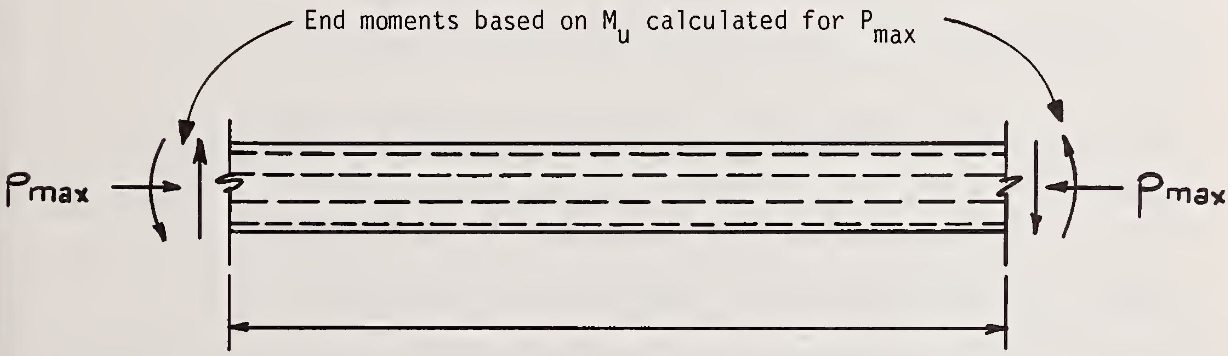


FIGURE C11-1



Note: Both end moments should be considered in both directions, clockwise and counterclockwise.

FIGURE C11-2

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COMMENTARY

CHAPTER 12: MASONRY

Sec.12.1 GENERAL

Chapters 12 and 12A define design and construction standards for masonry to provide resistance criteria consistent with the provisions of Chapters 1 through 6.

Chapter 12A provides basic design and construction criteria and is included since an adequate, comprehensive national standard covering all commonly used types of masonry construction is not available. Although Chapter 12A contains a few provisions for seismic resistance, it is modified by the provisions of Chapter 12 for three reasons:

a. Resistance is traditionally defined in terms of working stresses and working stress procedures, concepts not fully consistent with the load criteria of these provisions. Modification is accomplished by the provisions of Sec. 12.2.

b. Construction types and materials with lesser seismic resistance or inelastic capacity should not be used where the seismic risk is large. Therefore their use is controlled as a function of Seismic Performance Category (A through D) by the provisions of Sec. 12.3 through 12.6.

c. This format provides a mechanism for clearly indicating special seismic requirements that go beyond general good quality.

In this regard a short discussion about unreinforced masonry is appropriate. Although there are notable instances of unreinforced masonry construction behaving quite well when subjected to strong ground shaking, its general performance has been poor to disastrous. Some of the greatest examples of injury and loss of life have been associated with the failure of unreinforced masonry. These are strong arguments for not permitting its use at all when an adequate level of earthquake resistance is desired. However, Sec. 12.3 and 12.4 are quite permissive in allowing its use provided it is used only where an analysis for earthquake forces, when required, indicates it will work.

It is felt that in a large number and possibly a majority of situations where an analysis for earthquake forces is made, reasonable economic arrangements and sizes of unreinforced masonry will not satisfy the design standards and consequently the design will have to be altered to partially or fully reinforced masonry. One of the more typical situations where this will occur is in the load case of forces perpendicular to a wall where Formula 3-2a is required. It must be emphasized that, aside from the exceptions of Sec. 3.5, analysis for seismic forces is required and traditional proportioning rules, by themselves, are not adequate.

Sec. 12.2 STRENGTH OF MEMBERS AND CONNECTIONS

Strength of members and connections shall be based on Chapter 12A as modified by Chapter 12. Strengths are determined by conventional working stress procedures modified in a manner to more nearly reflect a resistance state comparable to yield for more ductile materials. Working stresses are increased using the ϕ factor concept. The following paragraphs provide an indication of the rationale behind the overall multipliers and the confidence levels associated with present working stresses.

With respect to the allowable stresses for seismic loads, compression and bearing allowable stresses are established by multiplying by 2.5 since the usual allowables are traditionally conservative. A multiplier of 2 is used for steel (for all purposes except shear) which reflects allowable versus yield of Grade 40 reinforcement. A higher value is not used for Grade 60 since this high-strength steel may not be fully effective when used in low-strength masonry. The multiplier is 1.5 for masonry tension through the units and zero through bed joints. (Note: Chapter 12A does not allow tension through stacked bond joints.) The unreliability of results on joint tension tests and the cyclic effect of seismic forces dictate the use of the zero ϕ factor for bed joints.

Multipliers for shear resistance of masonry, shear reinforcement, and bolts are set comparatively lower than other values. This reflects the variability and lack of test data (especially cyclic loading data) and indicates that some safety factors associated with present allowables are too low. In addition it provides a small increase in the resistance capacity, thereby minimizing rapid cyclic deterioration and premature brittle failure.

12.2.1 SPECIAL DESIGN PROCEDURES FOR UNREINFORCED MASONRY SUBJECTED TO SEISMIC FORCES

Two procedures for design are included in Chapter 12A. The modifications of this section are necessary to provide a rational basis for combined flexure and axial load design across bed joints consistent with the restriction of no tensile stress. The first modification applies to unreinforced masonry conventionally designed. It should be noted that bed joints in stacked bond construction are to remain uncracked as it is unrealistic to expect the masonry above the crack to be held up by a continuous vertical joint. Similar concepts underlie the second modification applicable to the optional design procedure.

Sec. 12.3 SEISMIC PERFORMANCE CATEGORY A

No special restrictions are placed on construction types and material for Category A construction.

Sec. 12.4 SEISMIC PERFORMANCE CATEGORY B

To increase its inelastic cyclic performance, structural masonry that is part of the seismic resisting system must be either partially or fully reinforced depending on the height of the structure. All stacked bond construction must be fully reinforced. This and other restrictions placed on stacked bond construction are a result of its poorer performance in past earthquakes as compared to that of running bond masonry. Further, it is unrealistic to expect the continuous vertical joint of stacked bond construction to be as effective in shear resistance as the keyed-together joints in running bond construction, especially when shrinkage and cyclic effects are considered.

UBC requirements for ties around anchor bolts are included. Shear wall requirements of Sec. 12.7 are made applicable to Category B and higher Category structures supplementing the requirements of Sec. 12A.6.4.

California Title 17 and 21 reinforcement provisions for screen walls are included supplementary to Sec. 12A.6.7. Joint reinformcement is considered to be effective for stress and strength for screen wall panels.

Seismic design is required for non-structural masonry due primarily to inertial forces. All such walls shall be anchored per Sec. 12A.4.7 and provisions shall be made to accommodate building drift.

Cavity wall construction is not allowed for structural masonry due to its sensitivity to inertial forces perpendicular to the plane of the wall.

Unburned clay masonry (adobe) and materials that are either especially brittle, weak, or of unreliable quality or durability are limited for structural use. They may be used for non-structural applications if analysis shows them to be adequate.

Sec. 12.5 SEISMIC PERFORMANCE CATEGORY C

Materials and construction types are more restrictive for these buildings. All masonry, structural or nonstructural, must be fully reinforced.

UBC requirements for column ties and tie hooks in seismic areas are included.

Required boundary shear wall elements when of masonry or concrete are limited to those with confinement or shall be structural steel.

Properly used joint reinforcement may be effective for crack control. Therefore it may be used to fulfill minimum reinforcement ratios. Because of the very small mortar cover it may receive and because experience with past earthquakes leads to questions about its effectiveness, it shall not be used to carry stress or establish strengths.

The construction requirements for stacked bond masonry are intended to improve the seismic performance of continuous head joints. Materials not permitted for structural use in Category B are additionally restricted for non-structural use in Category C. Several materials where reinforcement can not normally be incorporated are also restricted.

Sec. 12.6 SEISMIC PERFORMANCE CATEGORY D

This category demands the highest quality of construction practicable. Some of the requirements of Sec. 12.6 are presently in effect for schools and hospitals in California.

Sec. 12.6 includes criteria to improve the seismic performances of masonry in the following ways:

- Minimizing grout shrinkage, cracking, and separation from the units
- Avoiding reinforcement congestion and over-reinforcement to the extent of limiting effectiveness
- Providing for proper embedment of reinforcement in the grout
- Providing for proper grout flow and full consolidation
- Improveming stacked bond continuous head joint strength

Experience has shown that the quality desired for Category D masonry can only be achieved when adequate Special Inspection is provided. Requirements for Special Inspection are designated in Sec. 12A.7

Sec. 12.7 SHEAR WALL REQUIREMENTS

These requirements for seismic resistant shear walls are intended to improve inelastic performance. They supplement the provisions of Sections 12A.6.3(E) and 12A.6.4.

The reinforcement ratio is increased and the maximum reinforcement spacing is decreased to levels consistent with the inelastic demands on these members. The exception provides relief for the reinforcement ratios of lightly loaded members of the seismic resisting system, but not to the maximum spacing.

Intersections with cross walls or boundary elements when used, must be constructed the same as the walls themselves, (i.e., running bond) otherwise they must be constructed to develop the vertical shear at their junction to the web element. This vertical shear must be considered in design. Web reinforcing must be anchored into or fully developed at the boundary elements.

Where the structural system described in Sec. 3.3 and Table 3-B consists of an essentially complete vertical load carrying frame, boundary elements of the same construction as the frame columns must be provided for all shear walls. This includes requirements for columns in ductile structural frames when such frames are used. Required boundary elements must be designed to carry all the vertical loads (dead, live, and overturning), the web becoming nonbearing.

Although walls are commonly analyzed as beam-type elements for flexure and overturning, it should be recognized they are not relatively compact like beams but are composed of thin elements. Also their compression zones are not small, with a steep stress gradient, but generally under combined axial and flexural conditions, are large with a considerable length of the wall carrying a more or less uniform compression. Therefore the interaction formula, $f_a/F_a + f_b/F_b \leqq 1$, may not be used. Instead the total compression stresses under combined loading shall not exceed the allowable stress for axial compression of walls considering reductions for slenderness. An exception for pier type elements is provided in view of their generally short length. Shear reinforcing is required in both directions per the SEAOC Recommendations for concrete shear walls. This applies both to the wall panels themselves and to horizontal coupling elements not containing diagonal "X" bars.

Spacing of shear reinforcing, to be effective, shall not exceed one-third the width of the element.

COMMENTARY

CHAPTER 12A: MASONRY CONSTRUCTION

Sec. 12A.1 GENERAL

As stated in the commentary to Chapter 12, Chapter 12A is included in this document since an adequate comprehensive national reference standard covering all commonly used types of masonry construction was not available during the development of this document. The current situation with respect to masonry codes and standards is fragmented. Recognized regional Building Codes contain masonry chapters covering only those forms of construction common in their membership areas. Conflicting provisions sometimes occur. One example is the effective thickness of cavity walls. Similar situations exist with some ASTM Standards. For example, both ASTM C270 and C476 cover mortar. Mortar test procedures are covered (directly and by cross referencing) in ASTM C91, C109, C270, C476, and C780. Some of these are directed more toward evaluation of the cement than the mortar.

The current situation apparently results from the existence of many masonry industry associations each concentrating on their particular products as opposed to the situation that prevails in the concrete industry where there is one comprehensive national professional group and one trade group--the ACI and PCA. Design and construction standards emanating from the present industry associations such as "Building Code Requirements for Engineered Brick Masonry, SCPI 1969" and "Specification for the Design and Construction of Load-Bearing Concrete Masonry, NCMA 1968" are not comprehensive in scope. One of the most comprehensive and one of the few directed toward seismic problems is Chapter 24 of the Uniform Building Code. Chapter 12A used this as a base. (The UBC provisions were not adopted by reference since they are not a national standard.) Chapter 12A consists mostly of the 1976 UBC Chapter 24 and ICBO Standards in revised form. ASTM Standards are cited where appropriate. Additional material from California Titles 17 and 21 (for public schools and hospitals) is also included as well as items from industry standards. New material directed towards improving quality and seismic performance has been incorporated. This new material both modifies and supplements the UBC provisions.

Most of the new material included in Chapter 12A is judgemental and is based on the observation of construction practices and the performance of masonry in past earthquakes. Most of the significant masonry seismic research performed prior to the development of this document has either been included in the 1976 UBC provisions or in the provisions of Chapters 12 and 12A. Two major research projects are currently in progress at the Universities of California at Berkeley and San Diego. Unfortunately the timing was such that little or no information was available from these programs to include in this document.

Chapter 12A emphasizes construction quality because it is recognized that no form of construction is more susceptible to the effects of workmanship than masonry. Workmanship is impossible to define in a code and difficult, at best, to enforce in the field. Even when work is inspected, the inspector may not be sufficiently experienced to know what is required or he may be satisfied with only the locally prevailing minimum quality of workmanship which may be less than the code intent. It should also be recognized that, in areas not accustomed to seismic resistant construction, masonry workmanship and quality acceptable in nonseismic areas is not necessarily adequate for seismic resistant construction. It is hoped that the need for good workmanship and masonry quality will be made evident to engineers responsible for design and that they require that construction supervision be provided.

Chapter 12A is quite lengthy. It could be even longer had many local construction varients been included. It is assumed that the reader is or will become familiar with seismic design and the basic code provisions incorporated in the UBC and similar documents. Therefore the remainder of this Commentary will only touch upon the more significant items and deviations from the UBC masonry provisions. For those not familiar with the UBC provisions, detailed comparisons of Chapters 12 and 12A with codes they are familiar with is suggested.

12A.1.1 DEFINITIONS

AREA, NET VERTICAL SHEAR. Since in a shear wall there is vertical as well as horizontal shear, a definition for "net vertical shear area" is included as an aid in arriving at the area of critical sections.

BOND, RUNNING AND STACKED. "Running" and "stacked bond" are defined for enforcement purposes. The construction documents should clearly indicate the bond pattern with emphasis on corners and wall intersections. These may be constructed as stacked bond if not clearly shown and detailed where required.

EFFECTIVE ECCENTRICITY. "Effective eccentricity" is synonomous with "virtual eccentricity" of the SCPI Code.

STRUCTURAL AND NONSTRUCTURAL. The terms "structural" and "nonstructural" are added to enlarge upon the old concept of "bearing" and "non bearing". A shear wall is a structural element yet may not carry any superimposed vertical floor load. To ensure that the provisions are properly applied, all bracing elements, bracing systems and enclosure walls are considered structural, the latter in view of the hazard they present. Similarly the "seismic system" is defined to distinguish it from other structural elements.

12A.1.4 CRITERIA FOR MASONRY UNITS

The ASTM and other referenced standards define the intended usage of the various types, quality and grades of masonry units. Units intended for a nonstructural use shall not be used structurally. Units intended for a use in a moderate surrounding environment shall not be used in a more severe environment.

12A.1.11 GLAZED AND PREFACED UNITS

Specific ASTM Standards are not cited for glazed units since the surface glazing is not of structural importance. Glaze shall not occur on surfaces receiving mortar or grout.

12A.1.14 CEMENT

Note that Chapter 12 limits Masonry Cement to Category A Construction only and that it is not allowed by this section for grout. Blended hydraulic cements allowed for grout are those equivalent to the portland cement types permitted. Type IV cement is not included.

12A.1.16 MORTAR

ASTM C270 rather than C476 or a combination of both is used as the basic mortar specification in view of tradition, since it is a more flexible document and since the UBC provisions are written around it.

Types O and K mortar are not allowed in view of their very low strengths and in view of either their masonry cement or high lime content. The minimum required strength of Type N mortar prepared under the property specifications has been increased from 750 psi required by ASTM C270 to 1500 psi since UBC standard 24-22 requires the higher value for test samples. It is felt that when "Special Inspection" allowable stresses are used a strength of 1500 psi should be obtained. It should be noted that the workability of the mortar may be more important than its strength as workmanship can be excessively affected. Also mortar bond to masonry units has sometimes been known to decrease with higher mortar compressive strengths.

The UBC provisions have been rewritten to emphasize the two methods of mortar design--the property specification and the proportion specification. Due to the detrimental effects of some coloring admixtures and where field tests are required, only the property specification shall be used. A paragraph has been added to allow retempering consistent with good field practice. The subsection includes portions of ASTM C270 for convenience.

12A.1.17 GROUT

These provisions are those of ASTM C476 plus additional items.

Since many problems with grout are associated with shrinkage, larger proportions of coarse aggregate and larger aggregate sizes are allowed for large grout spaces. Coarse grout must be used for 5 inch and larger grout spaces. Maximum slumps are included to provide an upper bound on water and shrinkage and a lower bound to ensure flow. The lower bound slump of 4½ inches should only be used for coarse grout in very large grout spaces and only where consolidation and reconsolidation by mechanical vibration is provided. Consolidation is covered in Sections 12A.3.5 and 12A.3.6 and also Chapter 12.

Minimum strength is set at 2000 psi. It should be noted that higher strengths may be required to obtain high prism strengths. For instance, experience in California has indicated that to consistently obtain 3000 psi grouted hollow unit prisms using special high strength block, a grout strength of 4500 psi is often required.

Sec. 12A.2 CONSTRUCTION

Light fog spray wetting of concrete unit surfaces receiving mortar during excessively hot and dry weather will prevent rapid and excessive mortar water loss. Wetting shall not be excessive as this will compound the shrinkage problem for concrete units.

12A.2.1 JOINTS

Solid tight joints are a required quality and workmanship item. Shoved joints are required for all work. Special attention is required at joints with concrete. Roughening is required. Continuous vertical joints with concrete are a stacked bond situation unless keys are provided. The usual situation at floors where the top of slab is troweled smooth is not allowed.

12A.2.2 BOND PATTERN

The requirement for vertical head joints and horizontal bed joints is self-evident since many code provisions are based on this usual form of construction. Exceptions one and two are also self-evident. It should be noted that Chapter 12A does not explicitly consider arch construction - except for allowable compressive stresses. Exception three requires full consideration of the inclined joints in design and that such design be approved.

"Stacked" and "running" bond are defined in Sec. 12A.1.1. This definition, Section 12A.2.1, and this Section call attention to the conditions at corners and wall intersections. It is recommended that the design documents detail the bond pattern at corners and intersections if running bond is desired or the mechanical bond if stacked bond is desired.

Conditions other than running bond at vertical joints are considered stacked bond. When allowed, stacked bond may be used in unreinforced construction with mechanical bond consisting of joint reinforcing or where only corners and intersections are stacked bond, of metal rods or straps. Two No. 9 gage continuous wire reinforcing at 16 inches on center, traditional in most codes, is maintained as the basic minimum joint reinforcement.

C12A.2.2 Cont.

This is considered adequate for walls up to 8 inches thick with proportionately more required for thicker walls by the minimum reinforcement ratio of .00027, serving to prevent under-reinforcement of thick walls. Chapter 12 provides additonal requirements for stacked bond as well as Section 12A.3.7 and 12A.6.3(C).

These requirements do not prohibit the use of control joints along the length of walls or at corners and intersections provided they are accounted for in design. Similar true joints are recommended for nonstructural partitions along with contained "sliding" joints at their tops to provide for building drift. See Section 12A.2.6.

The requirements obviously do not restrict construction that appears to be stacked bond yet is not. Many hollow unit styles give this effect in running bond work.

Sec. 12A.4.9 provides that concentrated loads not be distributed across the continuous vertical joints of stacked bond construction as they may be ineffective in vertical shear. Vertical shear also exists in shear walls and similar reasoning would limit the use of stacked bond for these elements. However, except for Sections 12A.4.9 and 12A.6.3(C), Chapter 12A does not contain such a blanket requirement. Chapter 12 does provide limitations for its use or construction modifications for particular situations. Stacked bond is not recommended for shear walls especially those of unreinforced masonry. (For additional discussion see the Commentary to Sec. 12.4 of Chapter 12.)

12A.2.3 CORBELING

Corbels are critical elements and should be carefully analyzed, designed and detailed.

12A.2.4 REINFORCEMENT

Splice locations and lapped splice lengths should be clearly shown and/or scheduled on the design documents or approved by the design engineer. Note that the new splice requirements of Sec. 12A.6.3(D)7 preclude stating lap lengths for large bars as a simple multiple of bar diameter.

Note that many styles of joint reinforcement for hollow unit masonry being marketed may not provide the minimum mortar coverage when tooled joints are used and definitely will not when raked joints are used even though these requirements have been codified for many years.

Bar size is limited as the effectiveness of large bars in limited masonry grout spaces is unknown and questionable.

Welding is limited to weldable ASTM A706 reinforcing or to cases where the chemical constituents are known so that the provisions of AWS D12.1 can be properly applied.

12A.2.6 ANCHORAGE

Nonstructural components shall be anchored to the structure. To provide for building drift, it is recommended that non-structural partitions be anchored at their tops utilizing a contained "slip joint" detail allowing for movement in the plane of the wall and expansion-contraction type joints at their ends.

12A.2.7 BOLT PLACEMENT

Templates or similar are required for grouted-in bolts to prevent their movement during grouting. Bolts in reinforced work are required to be confined by reinforcement to help control the spalling that often occurs during earthquakes.

12A.2.8 PENETRATIONS AND EMBEDMENTS

Although frequently overlooked, it is self-evident that weakening elements be considered in design and constructed only as allowed by the design documents. Sometimes conflicts in the design documents occur when these elements are indicated on certain portions such as the mechanical, plumbing or electrical documents and not on the structural portions.

Sec. 12A.3 TYPES OF CONSTRUCTION

These are limited to the major types in the Subsections following. It is recognized that some forms or variations peculiar to limited areas of the country may be omitted. For these cases Sec. 1.5 will provide relief. However, they should not be more brittle or less capable of inelastic deformation than the types included on this document.

12A.3.4 CAVITY WALL MASONRY

This form of construction can be particularly sensitive to seismic inertial forces perpendicular to the plane of the wall and hence is not recommended. Its use is limited to Category A Construction. Where thermal insulation is desired, more modern and perhaps more economical alternatives should be explored. Where cavity walls are used, it is suggested that frequently spaced reinforced vertical elements be incorporated.

12A.3.5 GROUTED MASONRY

The UBC requirements have been amplified somewhat by the provisions of California Titles 17 and 21, and by some recommendations contained in "Handbook on Reinforced Grouted Brick Masonry Construction", Brick Institute of CA., Los Angeles, 1961, and by some minor new modifications. When constructed and reinforced in accordance with subsection 12A.3.5(C) the work can be considered "Reinforced Masonry".

The grout space widths given are a minimum. These may need to be enlarged to provide space between the reinforcement and the units for grout or for mechanical vibration. Although only required for Category D construction, it is recommended that the minimum grout spaces for reinforced work not exceed those of Sec. 12.6.1(A). A $\frac{1}{2}$ inch grout cover is required to provide for the 3/8 inch pea gravel of coarse grout and for proper consolidation.

Although only spelled out for high lift work it is essential that excessive mortar fins--over a 3/8 inch projection--not project into the grout space as they interfere with grout consolidation and initiate shrinkage cracking. For high lift work mortar droppings and other debris must be cleaned out. For low lift work this is also advisable however minor droppings can be worked into the grout by puddling if grouting follows the laying within a few minutes. Sand and hardened buildups should be avoided.

Keys or their equivalent are required at grout dams or barriers in walls that are part of the seismic system to provide for shear. Reinforcing shall not terminate at these locations. Obviously these requirements do not apply at control joints.

Puddling is not considered to be fully effective for grout consolidation in high lift work. Mechanical vibration including reconsolidation is required.

Masonry work using the higher stresses designated in Tables 12A.5 and 12A.7 as Special Inspection requires direct shear bond tests of cores to verify bond between units and grout. All high lift work requires cleanouts. Attention is directed to these items for appearance considerations.

Observation of construction and of earthquake performance indicate that confidence can be placed in well constructed reinforced grouted masonry. Although this is not evidenced by allowable stresses (as they conform to existing codes) it is evidenced by the comparitively few modifications for this work contained in Chapter 12.

12A.3.6 HOLLOW UNIT MASONRY

Hollow unit masonry can be particularly susceptible to poor workmanship for many reasons. One of these is the less than ideal situation that normally exists at the head joints; e.g., buttering with mortar only for the thickness of the face shells. This situation is aggravated when stacked bond construction is employed. Additional requirements are placed upon stacked bonding in Chapter 12 but apply only to reinforced construction. Unreinforced stacked bond structural elements, particularly shear walls are not recommended. Tight head joints can be obtained by paying proper attention to shoved joints, proper tooling and for concrete units, the shrinkage limitation of Sec. 12A.1.13.

Except at starter courses and cells to be filled, cross webs are not normally bedded in mortar, only the face shells. It is apparent then for ungrouted work that the face shells, the face shell bed joints and the face shell head joints must carry all the shear in shear walls and all compression stresses. The weakening effect of raked joints is unknown. However they must be deducted to arrive at net sections.

Bond between grout and the inside of the masonry units is often not achieved. One reason is the use of debonding compounds during manufacture of masonry units, a practice not properly controlled. Another is grout shrinkage. To help control shrinkage the following requirements should be noted:

- Coarse grout is required in the larger spaces by Sec. 12A.1.17(B)
- Reconsolidation is required by Sec. 12A.3.6(A)1 and 2
- Mechanical vibration is required for high lift work by Sec. 12A.3.6(A)2
- Larger grout spaces, coarse grout only, mechanical vibration only and special admixtures are required for Category D Construction by Sec. 12.6.1.

The lack of bond between units and grout has been noted in past earthquakes where face shells have spalled from the grout. The face shells apparently provides most of the resistance during initial stages. Lack of bond has also shown up in some low strength prism tests.

The grout shrinkage problem can be aggravated by the presence of mortar fins which initiate horizontal shrinkage cracks. Evidence of this cracking at a majority of bed joints has been noted in some samples cut from walls. Mortar fins are restricted by Sec. 12A.3.6(A).

Reinforced grouted hollow unit masonry is rarely more than one wythe thick. Therefore the bonding between wythes if this work is of multiple wythe construction is not covered. One method of accomplishing such bonding would be the cutting of sufficient interior face shells of both wythes to form grout connections common to both wythes.

Grouted unreinforced hollow unit work is not recognized in this document. Therefore grouting requirements are contained in Sec. 12A.3.6.(A). When fully conforming to that subsection the work can be considered Reinforced Masonry.

For high lift work, the total height of the grout lifts are limited for practical reasons such as inspection and the ability to achieve proper consolidation. Grout must be poured in partial lifts, preferably not exceeding four feet with consolidation and reconsolidation for each lift. Specific Inspection is required.

Horizontal reinforcement shall be placed in bond beam units. For partially grouted walls, this will facilitate proper top grout coverage. For solidly grouted walls, this will facilitate grout flow. Deep cut bond beam units are recommended.

The reasons for the grout cover requirements between the reinforcement and unit are similar to those for reinforced grouted masonry construction.

12A.3.7 PARTIALLY REINFORCED MASONRY

This may be designed as unreinforced masonry or the reinforced elements may be considered as providing strength if they fully conform to the design and construction requirements for reinforced masonry (including those for grout spaces, grout cover, grouting, etc.). To provide for this latter requirement and to provide for the effectiveness of the required reinforcement by construction details, partially reinforced construction is limited to grouted masonry and hollow unit masonry.

It is recognized that other forms of construction may be equally effective as those described in this section. For instance a modified form (closer spacing of reinforced elements) of the construction described in the South Florida Building Code utilizing concrete tie columns and beams may be acceptable. However, as previously explained, local variants are not included in this document.

Column reinforcement shall be the same as required for reinforced work. Vertical wall reinforcement spacing for thin walls shall not exceed 6 feet to provide reasonable unreinforced thickness ratios between vertical "studs". Similarly where distributed joint reinforcement is not provided, spacing of horizontal reinforcement is limited to avoid the large unreinforced areas that would otherwise occur in walls of large story height. To avoid under-reinforcement of thick walls a minimum reinforcement ratio is established. The ratio is approximately equal to what would normally occur with walls of usual proportions and also equal to traditional joint reinforcement consisting of two No. 9 gage wires at 16 inches on center in an eight inch thick wall. For stacked bond walls this ratio is increased to the minimum applicable for reinforced work.

For the reinforcement to be effective, splicing shall conform to the requirements for reinforced work.

Partially reinforced masonry is treated the same as unreinforced masonry for the R-factors of Chapter 3.

Sec. 12A.4 DETAILED REQUIREMENTS

12A.4.2 THICKNESS OF WALLS

Traditional empirical requirements are included and set forth in the text and Table 12A-2. For the most part these are contained in UBC and other codes with minor variations. The effective thickness of cavity walls and its maximum thickness ratio comes from "The Specification for the Design and Construction of Load-Bearing Concrete Masonry", National Concrete Masonry Association, Arlington, VA. This criteria appears more meaningful than other alternatives as it approximates the flexural strength of the equivalent section with the sum of the flexural strengths of both wythes, and also approximates the Euler buckling strength. For design purposes the effective thickness of cavity walls is redefined. See Sec. 12A.6.1(B).

Except for cavity walls, maximum thickness ratios are based on nominal thickness.

Since unreinforced masonry is particularly sensitive to inertial forces perpendicular to the plan of the wall, the exception emphasizes that liberalization of the requirements must be based on a thorough analysis including the items described.

The minimum nominal thickness of reinforced nonstructural walls and partitions is set at 4 inches due to the difficulty of properly reinforcing thinner construction.

It should be noted that the empirical requirements are compatible with the construction types included in Sec. 12A.3.1 through 12A.3.7. Special variants and their requirements could be approved under Sec. 1.5.

12A.4.9 DISTRIBUTION OF CONCENTRATED LOADS

The requirement restricting the distribution of concentrated loads across continuous vertical joints is extended to the consideration of overturning effects in stacked bond shear walls.

Sec. 12A.5 STRENGTHS AND ALLOWABLE STRESSES

12.A5.1 MASONRY

Compressive strength, f'_m, is established by prism tests before and during construction or assumed as listed in Table 12A-4. For the higher assumed strengths, verification is required either by testing the units or by furnishing acceptable certifications. For certifications to have validity, the required testing and plant inspection should preferably be done under the auspices of a recognized industry-wide program which includes frequent periodic monitoring. Special units (color, etc.) or runs should be tested. Strengths of grouted work exceeding 2000 psi cannot be assumed under the Table unless grout strength is verified by testing during construction. Tests of field made prisms are required to verify assumed strengths exceeding 2600 psi.

Prism, unit, grout and/or mortar tests by themselves do not qualify the work for "Special Inspection" stresses unless all the applicable Special Inspection and Testing requirements of Sec. 12A.7 are met.

Table 12A-4 is compiled from several sources with adjustments made for the prism height correction factor of the source as compared with that of Sec. 12A.8.1.

Allowable working stresses for unreinforced masonry designed by traditional procedures are listed in Table 12A-3. The value of f'_m need not be established, but must be verified. Masonry unit tests or certifications are required for the higher strength work. Axial compression stresses are only applicable if the members are not more slender than the ratios of Table 12A-2. For consistency with other requirements, allowable axial stresses for columns are 80 percent of those for walls. Reductions are made for lightweight concrete shear and tension.

In Table 12A-5 for reinforced masonry, some allowables for wall shear have been adjusted upward in view of the reinforcement ratio. The modulus of elasticity of 600 f'_m is a better approximation to test values (considering all the various masonry types) than 1000 f'_m as currently used. Tests indicate this quantity to be highly variable. No further refinements (square root functions) are justified. Lightweight concrete unit allowable shear is adjusted.

12A.5.2 STEEL

Allowable reinforcement stresses are the same for work with or without Special Inspection. However it should be recognized that reinforcing may not be fully effective if improperly placed, grouted, or spliced and completely ineffective if not grouted at all as sometimes happens.

12A.5.3 ANCHOR BOLT

Anchor bolt values are set forth in Table 12A-6. The reduction for work without Special Inspection is one-half in the NCMA Specification, one-third in the SCPI Recommended Practice, and zero in the UBC. Due to the brittle-type failure mechanism on overload, a factor of one-third is used here, however the values listed are for work without Special Inspection. Footnote two allows an increase for work having Special Inspection.

To fulfill the need, edge distances and spacings are established consistent with good practice.

Sec. 12A.6 DESIGN REQUIREMENTS

12A.6.1 DESIGN PROCEDURE FOR UNREINFORCED MASONRY

The procedure described herein relies on a conventional elastic analysis based on linear stress and strain distributions. Allowable stresses are those of Table 12A-3 with compression stresses only valid if the thickness ratios of Table 12A-2 are not exceeded. Column allowable compression is reduced 20 percent. Wythes of cavity walls and similarly constructed wythes of unreinforced walls where the collar joints are not solidly filled shall not be assumed to act compositly. The term "effective thickness" as used herein for stress analysis of cavity walls differs from the definition used in footnote 5 of Table 12A-2.

This design procedure is modified for seismic loadings by Sec. 12.2.1(A).

12A.6.2 ALTERNATE DESIGN PROCEDURE FOR UNREINFORCED MASONRY

Basically this Section includes those portions of the "Building Code Requirements for Engineered Brick Masonry", SCPI, 1969, that are incorporated into the UBC and which are applicable to unreinforced work. With proper constraints and allowable stresses the procedure could probably be made applicable to concrete, sand lime, and other clay units. Pending development of requirements, the restrictions of the SCPI document are maintained with regard to the use of clay units.

Portions of the SCPI document are incorporated into Sec. 12A.6.1 as they are applicable to the traditional design procedure for unreinforced masonry as well as this alternate procedure. Provisions of Sec. 12A.6.1 are applicable to the alternate procedure unless modified by those of Sec. 12A.6.2.

Allowable stresses are based on f'_{mb} instead of f'_m in order to preserve the allowable stresses of the SCPI document. Since prism strengths determined in the SCPI document use a different set of correction factors for prism height to thickness ratios than those of Sec. 12A.8.1, an adjustment is necessary. The 0.73 coefficient provides that adjustment.

This design procedure is modified for seismic loadings by Sec. 12.2.1(B).

(A) SLENDERNESS RATIOS. The term "slenderness ratio" differs from the term "thickness ratio" as used in Sec. 12A.4.2 and Table 12A-2. Although the maximum slenderness ratio formulas of the SCPI document are included, the maximum thickness ratios of Table 12A-2 shall not be exceeded unless all the criteria of the exception of Sec. 12A.4.2 are satisfied. Generally the thickness ratios will govern over the slenderness ratios and, considering inertial forces, analysis for forces perpendicular to the plane of the wall will govern over both the ratios.

Allowable stresses are listed in Table 12A-7. Compression and bearing stresses for work without Special Inspection are set at two-thirds of those with Special Inspection differing from SCPI which uses the same values and UBC which uses a factor of one-half. Two-thirds is justified considering the general conservatism of these allowables. Allowable tensions and maximum shears are similarly set using a factor of about one-half, the same as SCPI and UBC. Three-tenths the square root of f'_{mb} is used for the basic shear allowable per UBC rather than five tenths per SCPI. Minor adjustments are made for maximum shears using M and S mortar to correspond with similar maximums for reinforced work. Modulii of elasticity are lowered and correspond to those listed in Table 12A-5.

(B) SHEAR WALLS. The SCPI document's utilization of the dead load to increase allowable shear is maintained. However the maximum allowable shear, including this increase, is not to exceed the maximum listed in Table 12A.7. When considering seismic loads, the dead load shall be reduced by one-half in accordance with Formula 3-2a.

12A.6.3 DESIGN PROCEDURE FOR REINFORCED MASONRY

This is essentailly the old "working stress" procedure for concrete contained in ACI 318-56 and earlier standards as incorporated into the UBC in modified form applicable to masonry. Except for the development length concept and splices this version closely follows that of the UBC and is similar to the reinforced masonry provisions of the SCPI and NCMA documents. For younger engineers familiar only with "strength design" concepts for concrete, reference is made to the ACI standards mentioned above and to older reinforced concrete textbooks.

Although a strength design approach may be valid for reinforced masonry, the basic research necessary to establish both its validity and necessary constraints (maximum strain, etc.) has not been done. Such research is a prerequisite. It must include all the masonry variables such as clay units, concrete units, grouted masonry, hollow unit construction, etc. and must include load tests to verify strengths predicted by design procedures.

(C) SHEAR AND DIAGONAL TENSION. In stacked bond construction, the masonry shall not be assumed to resist shear. Stacked bond is not recommended for flexural members. If used, web reinforcement must cross the head joints, either as diagonal shear reinforcement or added longitudinal shear reinforcement.

(D) REINFORCEMENT DEVELOPMENT, ANCHORAGE, AND SPLICES. These provisions are essentially the development anchorage and splice requirements of ACI 318-71 as modified to be applicable to masonry. The flexural bond concept is dropped in favor of a development and anchorage concept (except as it is indirectly maintained by Formula 12A-10).

The basic development lengths, l_d, have been established from ACI 318-71 considering all horizontal bars to be "top bars". The situation with respect to vertical bars and their grouting is similar to top bars so that the 1.4 factor is considered applicable. Coefficients have been rounded. No further refinements are considered in view of the lack of reinforced masonry test data. Reductions are allowed only for reinforcement that probably will not be subject to full tensile stress by earthquake effects.

Allowable stresses at the start of hooks have been established, in simplified form, considering the ACI provisions as related to the allowable stresses herein. Lapped splice length requirements have been obtained from the basic development lengths described above and by considering all splices to be Class C per ACI 318-71. Again, no further refinements are justified in view of the lack of test data. Reductions are allowed only for certain columns. Application of the UBC bond provisions will result in lapped splice lengths for large bars that are quite a bit shorter than for similar bars in concrete of greater strength than masonry. This situation has been corrected by these new provisions.

Designers should note that the required splice and development lengths for large high-strength bars are quite long and might make their use impractical. Lacking test data as to their effectiveness, the use of vary large grade 60 bars is generally questionable. Lapped splice lengths for smaller bars will be close to the lengths obtained by use of the UBC bond stresses.

Welding of web reinforcement to the longitudinal bars is not allowed for anchorage. Such welding may be damaging to the main reinforcement.

(E) REINFORCED MASONRY WALLS. Traditional limitations for minimum wall reinforcement and maximum spacings are incorporated in Sec. 12.7 for walls. Additional requirements for shear walls are incorporated into Sec. 12A.6.4. Reinforcement ratios are increased and allowable spacings descreased by the provisions of Chapter 12 for seismic resistant construction.

(F) REINFORCED MASONRY COLUMNS. Additional tie requirements appear in Secs. 12.4(B), 12.5.1(B), and 12.5.1(C) for seismic resistant construction.

12A.6.5 MASONRY SHEAR WALLS

Shear walls are walls which resist in-plane shear with or without cross walls or other boundary elements serving as flanges.

Rules for effective flange widths are for cantilever-type walls. For wall elements acting in double flexure they may be too liberal as the height is considered from the roof downward. For double flexure, an effective height as a function of distance from the inflection points may be appropriate. Alternative determinations of effective flange width should consider shear-lag.

For unreinforced masonry, Formula 3-2a shall be used and will generally govern for the vertical stress case with overturning.

The often ignored shear and flexural effects in horizontal shear wall elements must be considered. Experience with past earthquakes and analysis indicates these elements, generally act as coupling beams between wall panels and are often the most highly strained portions of the system.

Inelastic cyclic tests of concrete coupling beams having short spans (span/depth less than two) indicate behavior that would not be expected from conventional reinforced theory. These and observations of earthquake damage lie behind the shear reinforcing requirements.

Only net areas, vertical and horizontal, shall be used in determining masonry shear resistance. Careful attention must be paid to hollow unit construction. Additional requirements for seismic resistant shear walls are contained in Chapter 12.

12A.6.7 SCREEN WALLS

This Subsection is taken from California Titles 17 and 21. Reinforcing provisions for seismic resistant construction are included in Sec. 12.4.1(D). Although unreinforced construction is allowed for Category A work, rational analysis may not be possible for some patterns. It is suggested that reinforcement be used.

Joint reinforcement is considered to be effective for stress and strength for screen wall panels.

Sec. 12A.7 SPECIFIC AND SPECIAL INSPECTIONS AND TESTS

As previously discussed, the quality of masonry, which is a hand-installed product, is quite sensitive to workmanship. Proper inspection can influence workmanship and can certainly call defective work to the attention of the proper authorities. However, experience shows that often those performing the inspections are not aware of what is required in the way of workmanship or aware of precisely what their duties are. This Section and portions of Sec. 1.6.2 attempt to fill the need in this area.

The masonry "inspector" is an individual who has taken special training, been examined for his knowledge, and is certified for same by a responsible authority. Preferably he should have also worked as a mason.

Inspection consists of more than just standing around watching the work. It consists of the inspector taking actions necessary to assure himself that the work is in compliance with the design documents and governing codes. The key word here is "action". The inspector should not be a passive observer. Moreover, he must review the design documents to familiarize himself with the special requirements for the particular job. These should be verified in a methodical manner.

Contrary to some codes, inspection for smaller jobs need not necessarily be "continuous" as long as all required items are periodically checked and as long as continuous inspection is not required by the Quality Assurance Plan. Additionally, unless "Special Inspection" is required, inspection need only extend to the Specific Inspections. However, if any defects are noted, regardless of whether or not they are included in the Specific Inspections, they must be reported.

Inspection for the purpose of obtaining the higher stresses permitted in Tables 12A.5 and 12A.7 for work designated as "Special Inspection" consists of all the listed items insofar as they are applicable to the work. Included are items frequently overlooked in such examination of materials or their certifications. For instance, grades and/or strengths of the units should be verified as well as their moisture conditions. Masonry units unaccompanied by proper certification require testing. Other required inspections, tests, and certifications are itemized. Unless all applicable items are provided, the work shall not be permitted to use "Special Inspection" allowable stresses. Providing only certain "Specific Inspections" and Tests is not the same as full "Special Inspection".

Testing agencies shall be under the responsible charge of a licensed or registered engineer (or engineers). Reports of work done by the agency should be signed by the(se) engineer(s).

Sec. 12A.8 TEST CRITERIA

This Section covers masonry test requirements not covered or inadequately covered by ASTM Standards. It is based on the UBC, UBC Standards, other sources and includes new material.

12A.8.1 MASONRY PRISMS

Although it has many defects the prism test procedure, as described in this Subsection, is retained as the Standard for defining assembly strength since no alternatives have been developed and since most allowable stresses rely on it. Size and h/d ratios constitute the major problem. Prisms supposedly represent the actual construction which more often that not consists of walls. However, prisms of low h/d ratios exhibit a different mode of failure than those with a high h/d ratio. Prisms with h/d ratios of 4 and above generally exhibit compressive failures typical of those observed in walls. Prisms with h/d ratios of three or lower do not generally exhibit this type of failure and this is attributed to the restraint provided by the platen during a prism test. The number of courses may be even more significant than the h/d ratios.

A related problem of the prism test exists with the strength correction factors for different h/d ratios. It appears that when normalized to a common base, the correction factors of Sec. 12A.8.1 and those from almost all other codes and standards in the world are nearly identical, all apparently derived from a common source which was limited in scope. It is clear that these factors must be clarified in future research.

Prisms cut from the actual work sometimes are quite different than the separately constructed specimens, both in terms of quality and strength. For instance, in grouted work, the grout consolidation and shrinkage is not the same in the prism as in the wall. Mortar fins seldom occur in prisms. Prism storage procedures [Sec. 12A.8.1(A)] generally provide better coring and cement hydration for the prisms than those obtained in the walls. To be truly representative, prisms should be cut from the work.

For hollow unit work, a weighted average of hollow and solid grouted prism strength shall be used to establish the strength for partially grouted work in view of probable strength difference between the two, attributable to variations of unit and grout strength and modulus of elasticity. The designer should also be aware that similar situations occur for columns and pilasters in grouted construction and that prisms are taken to represent the wall strength not the column strength. The ratio of grout area to the total area of columns can be quite different than similar ratios for walls. Barring verification by tests, the design should account for this.

The discussion above is included for perspective regarding our true knowledge of masonry strength. (For additional discussion, see "Mechanics of Reinforced Concrete Masonry: A Literature Survey", G. A. Hegemier, University of California, San Diego, Report No. AMES-NSF TR-75-5.) Furthermore, the test procedure, if it were fully valid, would be valid for wall compressive strength. Considering the above and the various forms of masonry, grouted masonry, unfilled hollow units, partial and solid grouted hollow units, etc., expressions such as "shear strength equal to a constant times the square root of prism strength" must be considered to be grossly approximate.

The number of test series is applicable separately to each different form of construction in the work.

12A.8.2 TESTS FOR GROUT AND MORTAR

Procedures for preparing mortar specimens are intended for field prepared sampling. Laboratory prepared mortar specimens are covered in the ASTMs. "Procedures" for preparing grout specimens are applicable to laboratory or field. Testing procedures for mortar and grout cover both laboratory- and field-prepared specimens.

12A.8.3 CORE TESTS FOR SHEAR BOND

Since no ASTM or other Standard is applicable, a core test procedure is specified to verify the bond between units and grout as specified in Sec. 12A.1.5 for grouted masonry construction. Tests are needed for work where "Special Inspection" is required.

Experience has shown that strengths are not sensitive to the test. Where bond is good, strengths considerably in excess of 100 psi are achieved. Where bond is poor, strengths are poor and often difficulties arise in obtaining cores without separation between units and grout during the coring process. For this reason, when separation occurs during the coring, the specimen shall tentatively be considered as failing the test; calling for futher investigation. The inspector should be present during coring.

In view of the above an elaborate test procedure and apparatus are not specified, only care to avoid any direct forces or significant flexure normal to the bond plane. The test is similar to those described in California Titles 17 and 21 and the "Handbook on Reinforced Grouted Brick Masonry Construction", Brick Institute of California, 1961.

COMMENTARY

CHAPTER 13: SYSTEMATIC ABATEMENT OF SEISMIC HAZARDS IN BUILDINGS

Sec. 13.1 GENERAL

There are many buildings in the United States that have primary structural systems which do not meet current seismic requirements. Such buildings might suffer extensive damage or even collapse if shaken by ground motion of an intensity that is considered likely for their locations. Thus, these buildings, which may be found in a high percentage among older buildings, may cause injury to their occupants, or people in the vicinity, in the event of an earthquake.

The number of buildings in the United States which do not meet minimum seismic standards is unknown. Estimates in the thousands have been made for Los Angeles (Los Angeles County, 1972) and similar estimates have been made for San Francisco (McClure, 1973). There are several reasons for the existence of such buildings:

A. Buildings that were designed prior to the introduction of reasonably adequate earthquake requirements into building codes and buildings that were not designed to resist earthquake forces.

B. Types of building construction that destructive earthquakes have shown are more vulnerable to earthquake forces than had been realized when they were designed and built.

C. Buildings in which the earthquake resistance has deteriorated due to factors such as: decreases in the strength of construction materials, fire damage, foundation settlement, alterations that have weakened structural elements, and damage sustained in past earthquakes.

13.1.1 IDENTIFICATION OF BUILDINGS REQUIRING EVALUATION

Identification of buildings not meeting minimum earthquake standards is necessary so that steps may be taken to reduce the hazard. However, there are major problems involved in such a course.

The social and economic impacts upon a community caused by the identification, evaluation and subsequent required repair or demolition of many buildings in a single area or neighborhood cannot be ignored. Obvious economic and physical hardships can occur unless the seismic hazard reduction program is carefully planned and aided by financial incentives and community participation.

Also, the program should be designed to focus upon the buildings apt to be most seriously deficient and most important to the community. Allowance must be made for the most efficient use of available technical and construction personnel and resources so that the program can result in significant hazard reduction within an acceptable time frame without serious disruption of the general economy.

In order to avoid having to evaluate the earthquake resistance of all existing buildings, a selection procedure has been developed to identify those classes of buildings most likely to contain nonconforming primary structural systems or nonstructural exterior walls, including parapets.

Seismic Performance Category C buildings selected for evaluation should undergo a relatively rapid evaluation which includes an on-site inspection. This Qualitative Evaluation classifies the primary structural systems and nonstructural exterior walls as being in general conformance with the provisions of this Chapter, not in conformance with the provisions of this Chapter, or uncertain as to conformance. Buildings in the latter two classifications must be strengthened, demolished, or must undergo an Analytical Evaluation.

Qualitative Evaluation is not considered sufficiently stringent for Seismic Performance Category D buildings so buildings in this classification requiring evaluation must be evaluated by the analytical method. Seismic Performance Category C buildings judged to be uncertain as to conformance with the provisions of this Chapter by Qualitative Evaluation must undergo an Analytical Evaluation. For the latter evaluation, analytical procedures are employed to determine the earthquake resistance that would be required if the buildings were to meet current code requirements. The Analytical Evaluation yields an earthquake capacity ratio which, together with the Occupancy Potential of the building, is used to determine if the building meets the provisions of this Chapter. The earthquake capacity ratio and the occupancy potential are then used to determined the length of time permitted to complete strengthening or demolition of nonconforming buildings. In this way, construction work for the abatement of earthquake hazards in existing buildings can be spread out over a long enough period to minimize economic and social disruptions.

Provisions of this section do not apply to the evalution of buildings immediately following an earthquake when it is necessary to decide quickly if damaged buildings are safe for continued occupancy. Procedures for this period are described in Chapter 15, Guidelines for Emergency Post-Earthquake Inspection and Evaluation of Earthquake Damage in Buildings.

Selected references and bibliography are listed at the end of this Commentary.

In order to achieve significant hazard reduction within an acceptable time, it is essential to identify those classes of buildings which are most likely to include the nonconforming buildings. With such an approach the majority of potentially nonconforming buildings will undergo evaluation without serious disruption to the social and economic life of the community.

There are existing buildings which were not designed for seismic forces at all, and others for which the seismic forces used in their design are now considered to be much too small. Earthquake regulations were introduced into the building codes of different communities at different times, and the first regulations in many instances are now considered totally inadequate. Thus, local building officials will have to determine the date at which reasonably adequate earthquake regulations were introduced into the building codes of their communities, and the buildings designed according to previous codes should be evaluated.

Available reference material, for example, indicates that the first reasonably adequate earthquake regulations were introduced into building codes in Los Angeles in 1933 and San Francisco in 1948. Thus, buildings in Los Angeles with Occupancy Potentials greater than 100 should be evaluated if they were designed in accordance with any code in effect prior to the 1933 edition of the Los Angeles City Building Code. Similarly, buildings in San Francisco should be evaluated if they were designed in accordance with any code in effect prior to the 1948 edition of the San Francisco City Building Code.

Since all existing buildings have been subjected to wind forces, they must have some lateral strength. In fact, in areas of low seismicity in which the expected ground motions have low intensity, the lateral strength required for wind forces is often greater than the lateral strength required for seismic forces. Thus areas not designated as having a Seismicity Index of 4 (Sec. 1.4) are not likely to contain a significant number of nonconforming buildings, and systematic evaluations need only be conducted in those areas designated as having a Seismicity Index of 4.

During the life of a building, the primary structural systems may have been weakened in many ways: by fire, earthquakes, foundation settlement, or major alterations. Such buildings should be evaluated even though designed after the enforcement of adequate seismic code regulations.

Destructive earthquakes in the past have shown that certain types of construction permitted by the first reasonably adequate and subsequent earthquake regulations perform poorly in earthquakes. Such types of construction are nonductile reinforced concrete frames and frames assembled from precast concrete elements, or modules without provision for adequate ductility. In addition, buildings with highly irregular plan configurations or discontinuities in vertical stiffness have proven to be vulnerable. Thus, buildings utilizing such types of construction or vulnerable features should also be evaluated even though they have been designed in accordance with the first reasonably adequate earthquake regulations or subsequent earthquake regulations.

Since the primary purpose of the evaluation is to prevent human injury and to protect life safety, buildings with low occupancy rates may also be exempted from evaluation. An occupancy rate could be defined as the number of occupant hours per day averaged over one year. However, the data for calculating occupancy rates would have to be obtained from building owners, managers, or tenants, and often the data would be subjective. The collection of such data would be time-consuming. For these reasons, it was decided to substitute Occupancy Potential for occupancy rate, since Occupancy Potential can be calculated quickly and objectively.

Occupancy Potential is calculated on the basis of the square feet per occupant (Table 13-A). The calculation is illustrated for some typical buildings in the following examples:

1. A dwelling of 3,000 square feet
   Square feet/occupant = 300 (Table 13-A)
   Occupant potential = 3,000/300 = 10

2. A 20-unit apartment building of 22,000 square feet
   Square feet/occupant = 200 (Table 13-A)
   Occupant potential = 110

3. A cinema of 4,900 square feet
   Square feet/occupant = 7 (Table 13-A)
   Occupant potential = 700

4. A department store consisting of a basement of 3,000 square feet, and a ground floor and second floor each of 6,000 square feet
   Square feet/occupant for basement = 20 (Table 13-A)
   Square feet/occupant for ground floor = 30 (Table 13-A)
   Square feet/occupant for second floor = 50 (Table 13-A)
   Occupant potential = 3,000/20 + 6,000/30 + 6,000/50 = 470

5. A grocery store of 2,100 square feet on the ground floor of a building above which there are two floors of apartments
   Square feet/occupant for grocery store = 30 (Table 13-A)
   Square feet/occupant for apartments = 200 (Table 13-A)
   Occupant potential = 2,100/30 + 2 x 2,100/200 = 91

6. A school building with 10,000 square feet of classroom space
   Square feet/occupant = 20 (Table 13-A)
   Occupant potential = 10,000/20 = 500

Buildings with low Occupancy Potential, less than 100, are exempt from evaluation regardless of construction. This will, in general, exempt one- and two-family dwellings and small conventional wood-frame buildings. The exemption does not apply to Seismic Performance Category D buildings.

A flow chart, Fig. C13-1, to aid the selection process and Form 1 (Fig. C13-2) for the collection of data on buildings in order to determine if they should be evaluated are provided.

13.2.1 QUALITATIVE EVALUATION

A Qualitative Evaluation of a primary structural system commences with a search for copies of the construction plans and design calculations for the building concerned. Possible sources for this documentation are:

1. Building owners (present, past)
2. Architects for the building
3. Structural engineers for the building
4. Building department
5. Contractors for the building
6. Shop drawings from materials fabricators and suppliers

If construction plans are available they should be studied to determine the primary structural systems for vertical loads and for lateral loads in both principal directions. A simplified sketch showing sizes of structural elements should be prepared for each primary structural system. The overall geometry of the structural systems should be checked for completeness and for discontinuities that might lead to stress concentration.

The inspection of the primary structural system consists of verifying that it meets the intent of the designer. The overall geometry of the structural system and the sizes of some structural elements should be checked against the sketches. In buildings in which the structural details are enclosed by finishes, the finishes should be removed in strategic locations.

If the construction plans are not available or are lacking in detail, which will generally be the case for older buildings, the primary structural systems must be identified at the time of the inspection. Again, sketches showing the sizes of structural elements should be made for each system. Preparation of the structural system sketches will take longer, but thereafter the evaluation will be the same as in the case where construction plans are available.

After the inspection of the primary structural systems has been completed, a decision should be reached as to whether or not each system is conforming to the provisions of this Chapter. The structural system should be classified as:

1. Conforming to the provisions of this Chapter

2. Not conforming to the provisions of this Chapter

3. Conformance cannot be determined by Qualitative Evaluation, and therefore an Analytical Evaluation is required.

In arriving at this decision the structural systems should be considered adequate unless there are serious deficiencies such as:

1. A primary structural system differs significantly from that shown in the construction plans.

2. A primary structural system cannot be defined or is incomplete or nonexistent.

3. A primary structural system has a discontinuity in stiffness along its height that would have a seriously adverse affect on its intended performance during an earthquake.

4. The building has a highly irregular layout in plan that would result in severe torsional forces during an earthquake.

5. Visual evidence of deterioration in the construction materials of the primary structural system.

6. The primary structural system has been significantly weakened by:

   a. Fire
   b. Earthquake
   c. Foundation settlements
   d. Alterations

7. The building employs only a reinforced concrete frame to resist lateral loads and the elements in the frame have not been designed to ensure adequate ductility.

8. The building employs a so-called "soft first story" which was designed before the disadvantages of this form of construction were appreciated.

9. Reinforced concrete columns are restrained by walls over part of their height so that the mode of failure in the columns during an earthquake will be shear rather than flexure.

10. The building employs masonry or reinforced concrete shear walls that lack sufficient steel reinforcement to ensure satisfactory performance during an earthquake.

11. The building employs precast concrete structural elements and the joinery details are inadequate.

Only the procedures to be employed in the Qualitative Evaluation of primary structural systems have been described. The procedures for nonstructural elements, although differing in detail, may employ the same principles. If the primary structural systems in Seismic Performance Category C buildings are adequate, then it is assumed that the nonstructural elements are also adequate or do not pose sufficient hazard in themselves to warrant evaluation. However, exterior nonstructural elements must be evaluated for all buildings in Category C regardless of Occupancy Potential, and all nonstructural elements must be evaluated in Seismic Performance Category D.

A flow chart showing a qualitative analysis is shown in Flow Chart 2 (Fig. C13-3).

13.2.2 ANALYTICAL EVALUATION

Procedures to be employed in the Analytical Evaluation of primary structural systems are described in this section. The evaluation procedures for nonstructural systems or components are generally analogous to those for primary structural systems; however, there are some significant differences. Standard analytical procedures have not been developed to the same extent for nonstructural systems or components as they have for primary structural systems. The difficulty in performing analytical evaluations of nonstructural systems and components will vary depending on the precision of the available analytical procedures. There are no analytical procedures for some types of systems and components, and therefore at the present time they cannot be evaluated by the analytical method.

The Analytical Evaluation will be based on the same techniques and assumptions that are permitted by the prevailing code for the design of new buildings. The significant differences between an analytical evaluation and a design for a new building are that, in the former, material properties cannot be specified but must be accepted as they are, and similarly there is no control over section properties or construction details.

(A) PRIMARY STRUCTURAL SYSTEMS FOR ANALYTICAL EVALUATION

In the case of Seismic Performance Category C buildings, sketches of the primary structural systems will have been made during the Qualitative Evaluation. Since qualitative analyses are not made for Seismic Performance Category D buildings, the sketches will not be available, but they may be prepared in the same way as for Seismic Performance Category C buildings. These sketches must be defined in complete detail for an Analytical Evaluation.

If construction plans are available, it should be relatively easy to define the primary structural systems in complete detail. In most cases, the Qualitative Analysis should have determined if the building has been constructed in accordance with the construction plans. If this check has not been made already, for example, in the case of Seismic Performance Category D buildings, it should be made at this stage.

If construction plans are not available, sketches must be prepared identifying the basic structural systems and details. In concrete or masonry structures, it will be necessary to determine the amount of steel reinforcement, which may be accomplished by removing sufficient material to expose the reinforcement at critical locations, by pachometer tests, or by X-rays. In removing concrete or masonry by sawing, chipping, or coring, care should be taken not to reduce the capacity of the system to support normal service loads.

(B) MATERIAL PROPERTIES

In order to make analytical evaluations, it is necessary to know the properties of the in situ construction materials. Often, in the case of steel and wood, it is possible to estimate strength conservatively.

For concrete and masonry, cores for testing should be obtained. A representative value for strength may only be obtained from the average of a number of tests. If only a few samples are tested, a value lower than the average should be assumed in order to be conservative. Sometimes, masonry and concrete have insufficient strengths to enable cores to be obtained. In such cases, conservative values for strength will have to be assumed.

(C) TOTAL LATERAL SEISMIC FORCE

The total lateral seismic shear force (V_{rs}) that an existing system or component would be required to resist in order to meet the requirements for a new building may be calculated using the analytical procedures permitted by the prevailing code applicable to the design of new buildings. The total lateral seismic force is evaluated by the formula:

$$V = C_S W$$

where

$$C_s = \frac{1.2\ A_v S}{R\ T^{2/3}}$$

Note: Refer to Sec. 4.2 for definitions of terms used in these equations.

The parameters in these equations should be evaluated as though the building were being designed. In the case of T, however, the fundamental period determined by an ambient vibration test may be used as an alternative to the computed period.

Some existing buildings may differ sufficiently from the types of buildings that are constructed currently so that none of the types listed in Table 3-B describe them precisely. Thus, it will be difficult to assign appropriate values to the seismic response modification coefficient R. In such cases, the type of building listed in Table 3-B closest to the existing building should be identified. The value of R given for this closest type of building should be taken as a guide to the appropriate value of R for the existing building. A higher value for R should not be taken without clear justification. Normally, a lower, more conservative value of R should be assumed.

The seismic shear force (V_{rs}) should be distributed over the height of the structual system using the appropriate distribution formula. The member forces and bending moments produced by these lateral forces may then be calculated to give bending moments (M_r), axial forces (P_r), and nominal shear stresses (v_r) at particular sections. In addition, the moments due to dead (M_D), live (M_L), and snow (M_S) loads have to be computed and the axial forces and nominal shear stresses as well.

(D) MEMBER CAPACITIES

The material and section properties may be used to determine the capacities of structural members and connections in the primary structural systems. The capacities to be determined are those for use in the strength design method. The procedures used in the calculation of capacities in the design of new buildings should be followed in the analytical evaluation of existing buildings, see Chapters 9, 10, 11, and 12. However, the capacity reduction factors used in the evaluation of existing buildings might be smaller than the corresponding ones used in the design of new buildings.

Construction practices, especially in the areas of detailing, have been improving over the years. In many existing structures the detailing practices might result in members and connections that have less strength than they would have if they had been constructed more recently. This difference may be taken into account by assigning smaller capacity reduction factors to members in existing buildings than for corresponding members in the design of new buildings. For reinforced concrete construction the capacity reduction factor for the design of new members in bending is 0.85, but a factor of 0.80 or smaller might be more appropriate for existing buildings. Since damage to reinforced concrete buildings in past earthquakes has often been precipitated by shear failures, significant reductions should normally be made in the capacity reduction factor for shear. The capacity reduction factor for shear might have to be reduced from the value of 0.6 used in the design of new buildings to 0.4 or lower for members in existing buildings. Connections in precast construction have been the source of damage to buildings in past earthquakes, so the capacity reduction factor for these connections might have to be reduced from the 0.5 for new construction to 0.4 or lower for existing buildings. Similar reductions in the capacity reduction factors will be necessary for members and connections in steel, masonry, and wood buildings. In all cases, the capacity reduction factor will be assumed according to the engineer's assessment of the detailing practices. However, values greater than those for new buildings may not be assumed.

In columns, where the bending moment capacity is a function of the axial load, the capacity should be evaluated under two loading conditions:

$$1.2Q_D + 1.0Q_L + 1.0Q_S \pm 1.0Q_E$$

and

$$0.8Q_D \pm 1.0Q_E$$

Using these procedures it will be possible to calculate section capacities in terms of moment (M_a), axial force (P_{CAP}), and nominal shear stress (v_{CAP}).

(E) EARTHQUAKE CAPACITY RATIO

Earthquake capacity ratios must also be evaluated for each of the two loading conditions specified above. The capacities available to resist earthquake forces at particular sections may be computed in terms of moment for these loading conditions as:

$$M_A = M_a - (M_D + M_L + M_S)$$

or

$$M_A = M_a - M_D$$

Generally, the moment capacity M_a will differ for the two loading conditions.

Similar available capacities in terms of nominal shear stress and axial force may be computed as follows:

$$v_A = v_a - (v_D + v_L + V_S)$$

or

$$v_A = v_a - v_D$$

and

$$P_A = P_{CAP} - (P_D + P_L + P_S)$$

or

$$P_A = P_{CAP} - P_D$$

Earthquake capacity ratios may be calculated in terms of moment for various sections by dividing the available capacity by the appropriate moment obtained for that section in the structrual analysis under earthquake loads. This ratio is in the form M_A/M_r, and similar ratios v_A/v_r and P_A/P_r may be obtained for shear stresses and axial forces, respectively. If these ratios at all sections are equal to or greater than unity, the building meets the prevailing code strength requirements, and, if not, the smallest ratio is considered to reflect the earthquake capacity of the building, and this ratio is designated r_c.

In the above discussion it was tacitly assumed that strength and not drift limitations should be checked. Drift due to earthquake loading may be calculated for existing buildings in the same manner as drifts are calculated in the design of new buildings. If the ratio of allowable drift to computed earthquake drift is greater than unity, drift limitations are satisfied. If this ratio is less than unity, drift limitations are not satisfied, but they only govern the earthquake capacity of the building if the drift ratio is less than the smallest force or moment ratio at any section. When drift limitations govern, the earthquake capacity ratio, r_c, is equal to the drift ratio.

Deflection amplification factors, C_d, are employed in calculating earthquake drifts and appropriate values are given for different types of construction in Table 3-B. If none of the types of construction listed in Table 3-B fits the building under evaluation, the C_d factor for the type that fits the building closest may be used as a guide to estimate the appropriate factor for the building. The value obtained from the table or a higher, more conservative, value should be assumed.

(F) MINIMUM ACCEPTABLE EARTHQUAKE CAPACITY RATIOS

If the earthquake capacity ratio (r_c) of an existing building determined by analytical evaluation is less than unity, the building does not satisfy the earthquake provisions of the prevailing code. Values less than but close to unity probably indicate that the building is only a slight hazard, whereas values close to zero indicate that extreme hazards may exist. It is necessary to define a minimum value for earthquake capacity ratio that will distinguish buildings with acceptable hazards from those with unacceptable hazards. Buildings with earthquake capacity ratio less than this minimum acceptable ratio must be strengthened or demolished, but those with ratios greater than the minimum are considered acceptable hazards.

The minimum acceptable earthquake capacity ratio must balance the hazard against the cost to repair the hazard. The hazard depends on both the function of the building and the potential number of occupants. Thus, it is necessary to develop two minimum acceptable ratios, the first for buildings in Seismic Performance Category D and the second for buildings in Seismic Performance Category C. For buildings in Category D, a minimum acceptable earthquake capacity is $r_c = 0.5$. For buildings in Category C, a minimum acceptable earthquake ratio is:

$$r_c = 0.25\left(1 + \frac{OP-100}{700}\right) \text{ but not greater than } 0.5$$

where OP is the occupancy potential

Since r_c need not be taken greater than 0.5, buildings with occupancy potentials of 800 or greater have the same minimum acceptable earthquake capacity ratio as Category D buildings. The minimum acceptable ratios for earthquake capacity for Category C buildings are shown as functions of Occupancy Potential in Fig. C13-5.

Generally, the minimum earthquake capacity ratios defined above apply to both the primary structural systems and the nonstructural components associated with a building. However, nonstructural walls and attachments (including parapets) to structural walls that pose a hazard to people at the exterior of Category C buildings have been made an exception. The minimum acceptable earthquake capacity ratio for these exterior nonstructural components is 0.5.

A flow chart showing the steps in Analytical Evaluation is shown in Fig. C13-4.

Sec. 13.3 HAZARD ABATEMENT MEASURES

Strengthening a primary structural system in an existing building will probably result in an increase in the building's life expectancy. However, the life expectancy is still probably less than the life expectancy for a similar new building. This shorter life expectancy suggests that an existing building should not have to comply fully with the seismic provisions for new buildings, but since the life expectancy of both new and existing buildings is so uncertain, it is impossible to allow for this factor in strengthening requirements.

If a primary structural system or nonstructural component requires strengthening, then it is desirable that it be brought up to full compliance with the prevailing code. Full compliance (earthquake capacity ratios equal to or greater than 1.0) is required for Category D buildings and Category C buildings that have Occupancy Potentials of 800 or greater. Full compliance is also required for interior nonstructural components in Category D buildings and for exterior nonstructural components in all buildings. For Category C buildings with occupancy potentials less than 800, the full compliance requirement is relaxed pro rata as the occupancy potential decreases until a building with an Occupancy Potential of 100 need only be strengthened so as to achieve an earthquake capaity ratio equal to or greater than 0.5. These strengthening requirements for Category C buildings are illustrated in Fig. C13-6.

The act of strengthening the structural systems in an existing building in order to abate a hazard may increase the total lateral seismic force requirement. For example, the addition of a shear wall to a building will shorten the building period and thus probably increase the induced seismic forces during an earthquake. The building must be provided with sufficient strength to resist the seismic forces computed for its strengthened state and not the seismic forces based on its state before strengthening.

Emphasis has been placed upon strengthening of primary structural systems. Special attention should also be given to such details as diaphragm chords, collectors, precast element interconnections to frames or supports, anchorages of exterior building skins and all other details associated with tying the building together.

Methods of strengthening buildings are described in Chapter 14, Guidelines for the Repair and Strengthening of Existing Buildings.

Some historical buildings may not be able to meet all of the strengthening requirements of these provisons without loss of the character which distinguishes them. In such instances, every effort should be made to find reasonable alternatives to these provisions that will allow for strengthening while preserving the historical character of the buildings. If necessary, special commissions may be empaneled to study proposed alternative solutions.

13.3.2 PERMISSIBLE TIME TO COMPLETE SEISMIC HAZARD ABATEMENT MEASURES

It is possible, if the rate at which buildings are evaluated is sufficiently high, that more buildings will be declared hazardous than could be strengthened without causing serious economic disruptions or overtaxing the construction industry. In order to avoid these problems, the work must be scheduled so that buildings are strengthened in the order of diminishing hazards. The owner is permitted a length of time to complete strengthening work that depends on the buildings earthquake resistance ratio, function, and occupancy potential. If the owner decides to demolish the building instead of strengthening it, he is permitted the same length of time to complete the demolition.

For Seismic Performance Category D buildings, the length of time in years permitted to strengthen or demolish a building is:

$$t_x = \alpha_t r_c$$

and t_x need not be less than one year. For Seismic Performance Category C buildings, the length of time in years permitted to strengthen or demolish a building is:

$$t_x = \alpha_t \left(1 + \frac{200}{OP}\right) r_c$$

The value of α_t is to be determined by the Regulatory Agency. If it intends to have all buildings in its jurisdiction evaluated in a short time, the α_t should be made relatively small. Conversely, if the buildings are evaluated slowly, so that the rate of evaluation will avoid economic and construction problems, α_t may be made relatively large. The value of α_t need not be the same for Category D buildings and Category C buildings. A typical value for α_t may be 12, and for this value t_x is plotted against r_c in Fig. C13-7 for Category D buildings and for Category C buildings with Occupancy Potentials of 100 and 800. r_c is the least value of earthquake capacity ratio computed for a primary structural system in a building. For buildings declared nonconforming by Qualitative Evaluation, r_c shall be 0.1.

The minimum acceptable earthquake capacity ratios for interior nonstructural elements are the same as those for primary structural systems of the building. If the primary structural system is to undergo strengthening, the permissible time for strengthening or removal of nonstructural elements, other than exterior nonstructural elements, is the same as for the strengthening of the primary structural system. Thus, the strengthening of primary structural systems and nonstructural elements can be accomplished simultaneously.

Since the hazards associated with external nonstructural elements are considered to be particularly severe, the permissible time to strengthen or remove inadequate exterior nonstructural elements is limited to one year. Nonconforming interior nonstructural elements in Category D buildings with conforming structural systems must be strengthened or removed within a period of one year.

FLOW CHART 1 : SELECTION OF BUILDINGS FOR EVALUATION

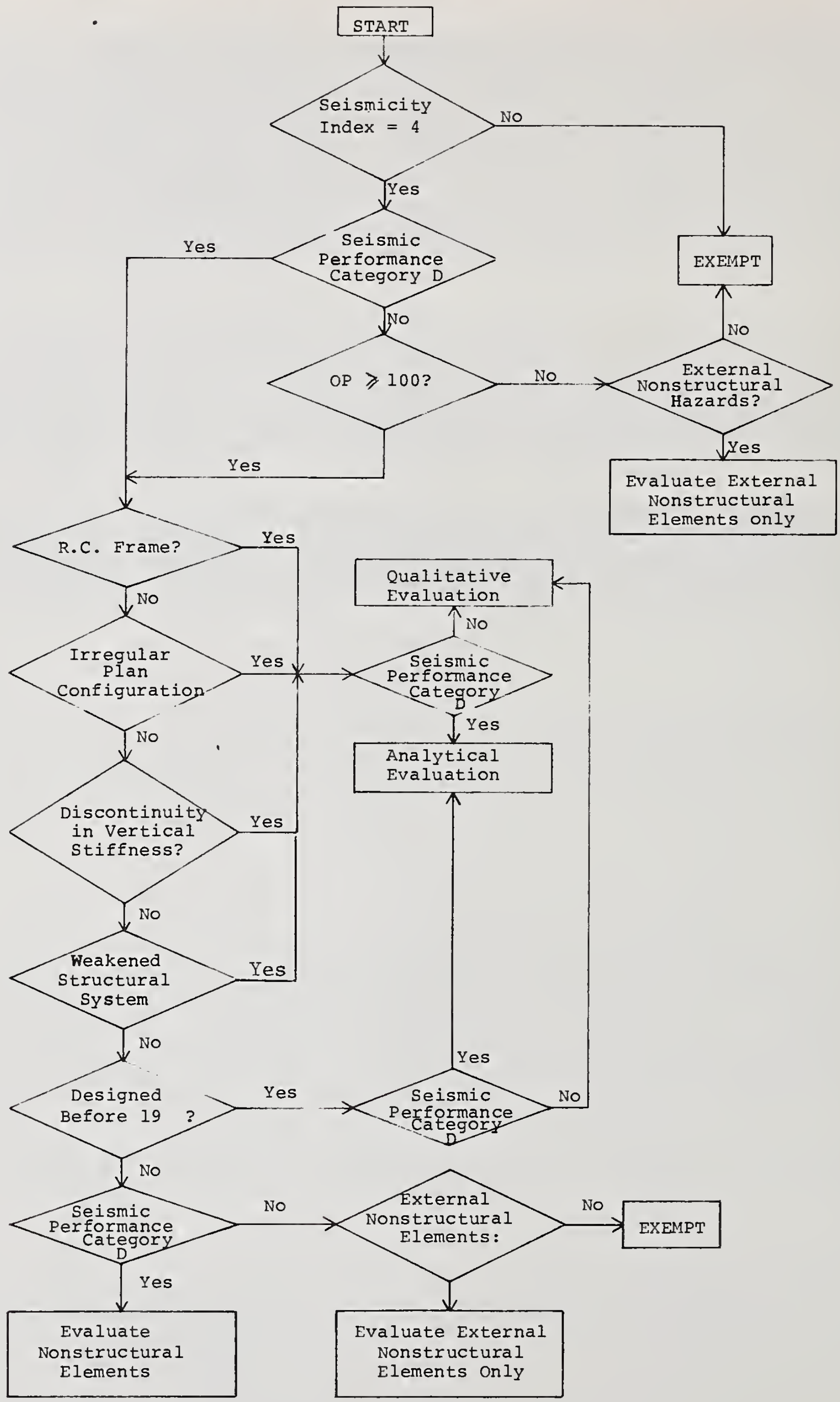

FIGURE C13-1

DATA FORM FOR SELECTION OF BUILDINGS FOR EVALUATION

Building Name: ______________________________

Building Address: ______________________________

Seismicity Index: ______________

Code used in design: ______________________________

Design date: ______________

Construction date: ______________

Building Use:

SP Category D	SP Category C
Fire Station	Theater
Police Station	Stadium
Hospital	School
	Store
	Office
	Hotel
	Apartment
	Dwelling
	Factory
	Garage
	Other ______________

No. of Stories: ______________

Occupancy Potential: (Not required for Seismic Performance Category D buildings)

Floor	Number	Area	Sq. ft./ Occupant	Occupant Potential
Basement				
Ground Floor				
Floor Type 1				
Floor Type 2				
Floor Type 3				
Floor Type 4				

Total =

FIGURE C13-2

(page 1 of 3)

DATA FORM FOR SELECTION OF BUILDINGS FOR EVALUATION

Does the building have a highly irregular plan configuration that might result in severe torsional response during an earthquake?

Yes ☐ No ☐

Is there a severe discontinuity in stiffness along the height of the building?

Yes ☐ No ☐

Are there exterior nonstructural elements such as walls and parapets that might collapse during an earthquake and if so do these elements pose a significant hazard to public safety?

Yes ☐ No ☐

Are there interior nonstructural elements that might collapse or malfunction during an earthquake and if so do these elements pose a significant hazard to public safety?

Yes ☐ No ☐

CONCLUSIONS (path leading to conclusion is indicated on Flow Chart, page 6a)

1. Building is exempt from evaluation ☐
2. Building requires qualitative evaluation ☐
3. Building requires analytical evaluation ☐
4. Nonstructural elements (internal and external) require evaluation ☐
5. Only external nonstructural elements require evaluation ☐

FIGURE C13-2

(page 2 of 3)

DATA FORM FOR SELECTION OF BUILDINGS FOR EVALUATION
Primary Structural Systems for Lateral Loads

STRUCTURAL SYSTEM	VERTICAL SEISMIC RESISTING SYSTEM	N-S	E-W
BEARING WALL SYSTEM: A structural system without an essentially complete vertical load carrying space frame. Seismic force resistance is provided by shear walls or vertical trusses.	Wall panels with light framing	☐	☐
	Reinforced concrete or masonry shear walls	☐	☐
	Vertical bracing trusses	☐	☐
	Unreinforced masonry	☐	☐
BUILDING FRAME SYSTEM: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or vertical trusses.	Wood sheathed shear panel	☐	☐
	Reinforced concrete or reinforced masonry shear walls	☐	☐
	Vertical bracing trusses	☐	☐
	Unreinforced masonry	☐	☐
MOMENT RESISTING SPACE FRAME SYSTEM: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by a moment resisting space frame designed for the total prescribed seismic forces	Ductile structural frame	☐	☐
	Structural steel frame	☐	☐
	Reinforced concrete frame	☐	☐
DUAL SYSTEM: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided primarily by shear walls or vertical trusses. A ductile moment resisting space frame capable of resisting at least 25 percent of the prescribed seismic forces shall also be provided.	Reinforced concrete or reinforced masonry shear walls	☐	☐
	Wood sheathed shear panel	☐	☐
	Vertical bracing trusses	☐	☐
SPECIAL STRUCTURES:			
Canopies and walkways with space frame resisting total prescribed seismic forces and providing support for vertical loads.	Structural steel frame	☐	☐
	Ductile frame	☐	☐
Inverted pendulum structures with the framing resisting the total prescribed seismic forces and providing support for vertical load.	Structural steel frame	☐	☐
	Ductile frame	☐	☐

FIGURE C13-2

(page 3 of 3)

FLOW CHART 2 : QUALITATIVE EVALUATION

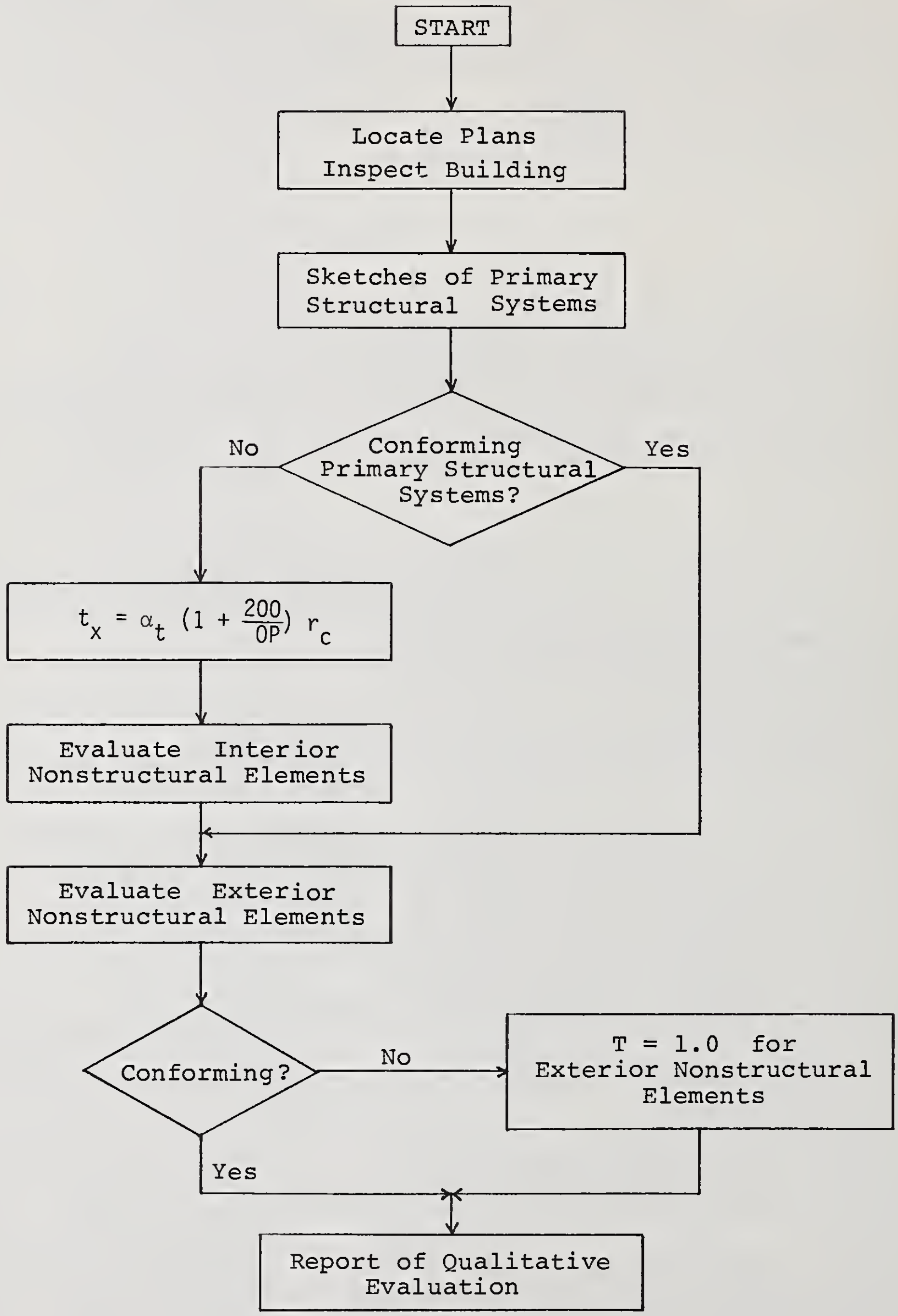

FIGURE C13-3

FLOW CHART 3 : ANALYTICAL EVALUATION

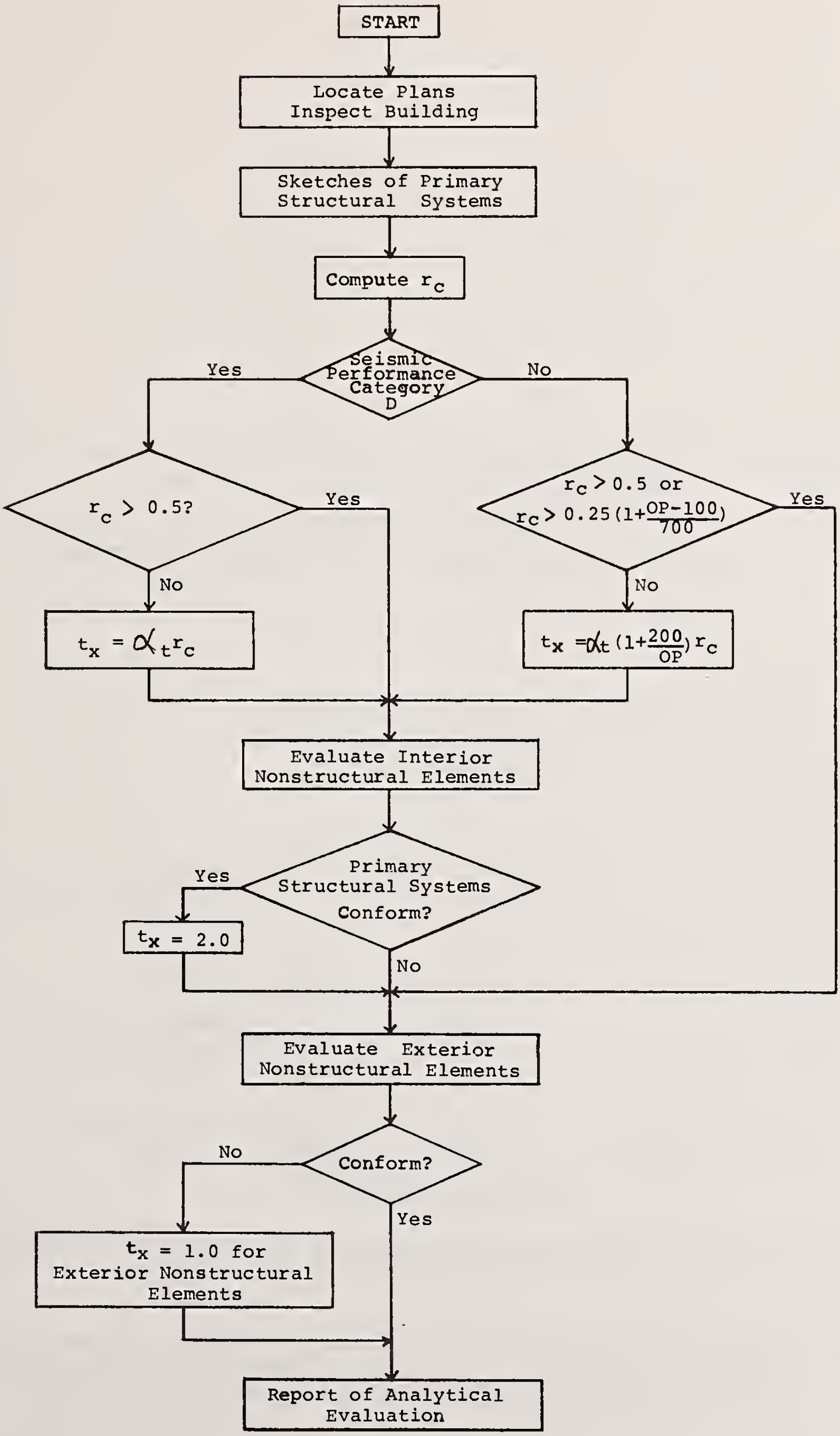

FIGURE C13-4

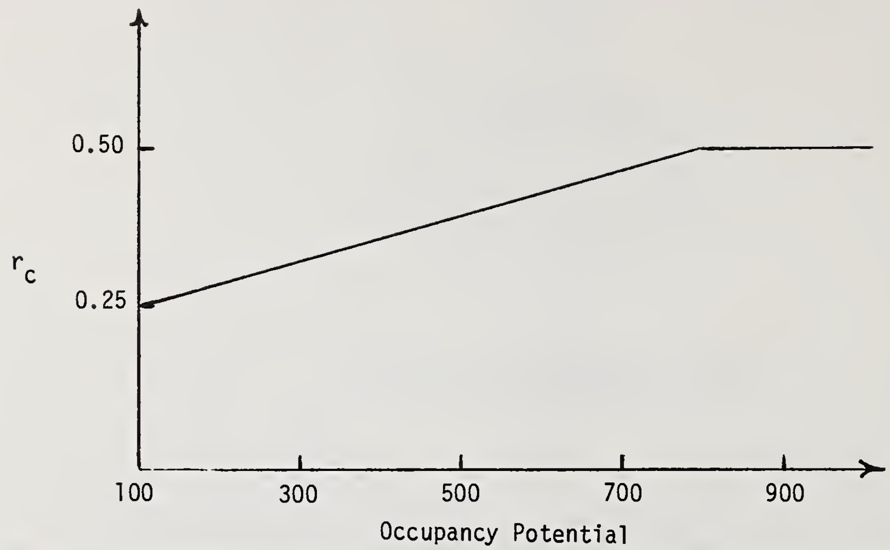

FIGURE C13-5 MINIMUM ACCEPTABLE EARTHQUAKE CAPACITY RATIOS FOR CATEGORY C BUILDINGS

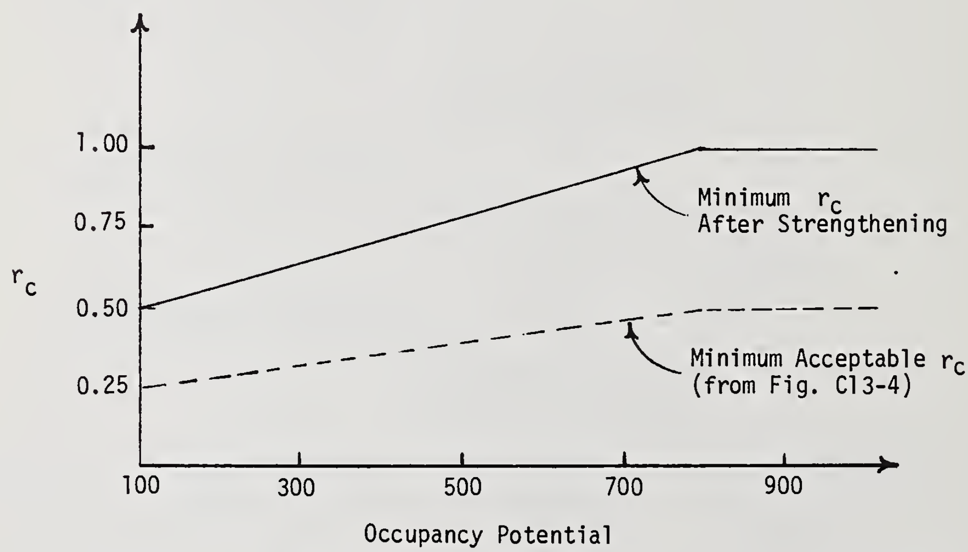

FIGURE C13-6 STRENGTHENING REQUIREMENTS FOR CATEGORY C BUILDINGS

Note: The values in Figures C13-5 and C13-6 are the best judgment of the Committee from information available. Application of these values in test structures may indicate that adjustments are desirable.

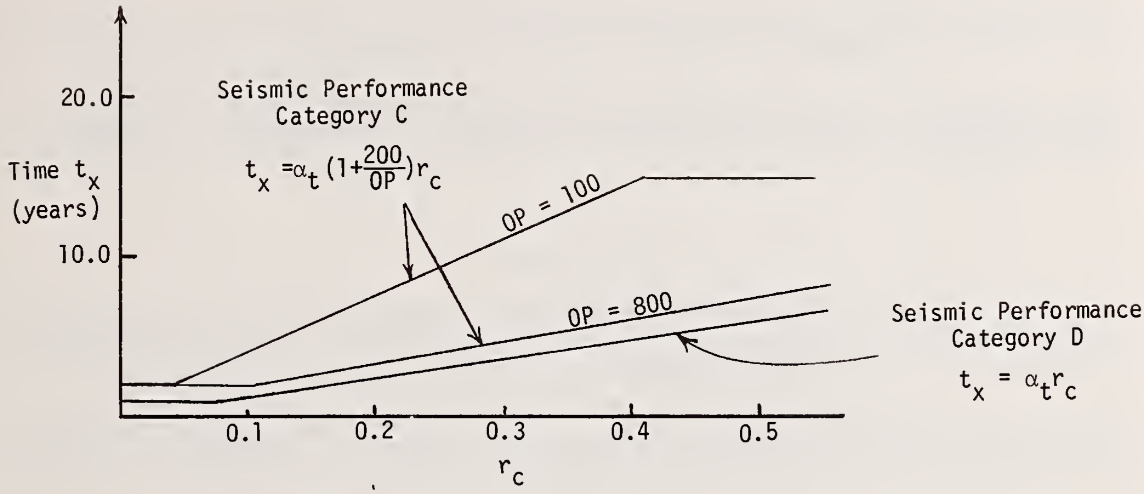

FIGURE C13-7 TIME TO STRENGTHEN OR DEMOLISH BUILDING IF $\alpha_t = 12$

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APPENDIX A

ATC-3 PROJECT PARTICIPANTS

APHABETICAL LISTING - SEE INTRODUCTION
FOR PROJECT ORGANIZATION

Dr. Mihran S. Agbabian
Agbabian Associates
250 North Nash Street
El Segundo, CA 90245

Dr. S. T. Algermissen
U.S. Geological Survey
2504C Stop 978
Denver Federal Center
Denver, CO 80225

Dr. Ignacio Arango
Woodward-Clyde Consultants
Embarcadero Center, Suite 700
San Francisco, CA 94111

Mr. Thomas G. Atkinson
Atkinson, Johnson & Spurrier
4121 Napier Street
San Diego, CA 92110

Mr. J. Marx Ayres
Ayres & Associates
1180 So. Beverly Drive
Los Angeles, CA 90035

Mr. Stephenson B. Barnes
S. B. Barnes and Associates
2236 Beverly Boulevard
Los Angeles, CA 90057

Dr. Jack R. Benjamin
Engineering Decision Analysis, Co., Inc.
480 California Avenue, Suite 301
Palo Alto, CA 94306

Professor Glen Berg
Department of Civil Engineering
University of Michigan
Ann Arbor, MI 48104

Professor V. V. Bertero
Department of Civil Engineering
University of California
Berkeley, CA 94720

Professor Jacobo Bielak
Instituto de Ingenieria
Universidad Nacional
Autonoma de Mexico
Mexico 20, D.F., Mexico

Mr. Warren E. Blazier, Jr.
Bolt, Beranek and Newman, Inc.
120 Howard Street
San Francisco, CA 94105

Dr. John A. Blume
URS/John A. Blume & Associates, Engineers
130 Jessie Street
San Francisco, CA 94105

Professor Bruce Bolt
Seismographic Station
475 Earth Science Building
University of California
Berkeley, CA 94720

Mr. Elmer E. Botsai
321 Wailupe Circle
Honolulu, HI 96821

Professor Boris Bresler
570 Vistamont Avenue
Berkeley, CA 94708

Mr. Walter A. Brugger
1323 Van Dyke Road
San Marino, CA 91108

Mr. Vincent Bush
International Conference
of Building Officials
5360 So. Workmans Mill Road
Whittier, CA 90601

Mr. James Cagley
Martin & Cagley
6000 Executive Boulevard
Rockville, MD 20852

Professor Anil K. Chopra
Department of Civil Engineering
University of California
Berkeley, CA 94720

Professor Ray W. Clough
Department of Civil Engineering
University of California
Berkeley, CA 94720

Dr. Charles E. Culver
Office of Housing & Building Technology
Center for Building Technology
National Bureau of Standards
Washington, DC 20234

Dr. Jagat S. Dalal
United Engineers & Constructors
1401 Arch Street
Philadelphia, PA 19105

Mr. Henry J. Degenkolb
H. J. Degenkolb & Associates
350 Sansome Street
San Francisco, CA 94104

Mr. Walter L. Dickey
1014 Fortune Way
Los Angeles, CA 90042

Mr. Shaefer J. Dixon
Converse, Davis & Associates
126 West Del Mar Boulevard
Pasadena, CA 91105

Dr. Neville C. Donovan
Dames & Moore
500 Sansome Street
San Francisco, CA 94111

Mr. James Dowling
AIA National Code Center
1735 New York Avenue N.W.
Washington, DC 20006

Mr. Eric Elsesser
Forell/Elsesser Engineers, Inc.
1005 Sansome Street
San Francisco, CA 94111

Professor Steven Fenves
Department of Civil Engineering
Carnegie Mellon University
Shenley Park
Pittsburg, PA 15213

Mr. John Fisher
Skidmore, Owings and Merrill
One Maritime Plaza
San Francisco, CA 94111

Mr. John W. Foss
Telephone Buildings and Equipment Dept.
Bell Laboratories
Whippany, NJ 07981

Mr. William E. Gates
Dames and Moore
1100 Glendon Avenue
Los Angeles, CA 90024

Mr. Alfred Goldberg
44 Greenwood Bay Drive
Tiburon, CA 94920

Mr. Leslie W. Graham
Graham & Kellam
55 New Montgomery
San Francisco, CA 94105

Mr. Norman R. Greve
Greve & O'Rourke
8055 Overland Avenue
Los Angeles, CA 90034

Mr. Karl Guttman
Kasin, Guttman & Associates
594 Howard Street
San Francisco, CA 94105

Professor William J. Hall
Department of Civil Engineering
1245 Civil Engineering Building
University of Illinois
Urbana, IL 61801

Professor Robert D. Hanson
Civil Engineering Department
University of Michigan
Ann Arbor, MI 48104

Mr. Richard L. Hegle
2184 Halekoa Drive
Honolulu, HI 96821

Professor George W. Housner
California Institute of Technology
4084 Chevy Chase Drive
Pasadena, CA 91103

Mr. Warner Howe
Gardner and Howe Engineers
1255-A Lynnfield Road, Suite 194
Memphis, TN 38138

Dr. I. M. Idriss
Woodward-Clyde Consultants
Two Embarcadero Center, Suite 700
San Francisco, CA 94111

Professor Paul C. Jennings
Thomas Lab, Mail Code 104-44
Division of Engineering and Applied Science
California Institute of Technology
Pasadena, CA 91109

Mr. Roy G. Johnston
Brandow & Johnston Associates
1660 West 3rd Street
Los Angeles, CA 90017

Mr. Gerald H. Jones, P.E.
BOCA Representative
8500 Santa Fe Drive
Overland Park, KS 66212

Mr. John Kariotis
Kariotis & Kesler
1414 Fair Oaks Avenue
South Pasadena, CA 91030

Mr. H. S. Kellam
Graham & Kellam
55 New Montgomery
San Francisco, CA 94105

Mr. Earnest Kiker
Southern Building Code Congress
3617 Eighth Avenue South
Birminigham, AL 35222

Professor Henry Lagorio
Department of Architecture
University of Hawaii
Honolulu, Hawaii 96822

Professor Kenneth L. Lee
Department of Civil Engineering
University of California
Los Angeles, CA 90024

Mr. Melvyn H. Mark
Ferver Engineering Company
3487 Kurtz Street
San Diego, CA 92110

Dr. Ronald L. Mayes
Applied Technology Council
480 California Avenue, Suite 205
Palo Alto, CA 94306

Dr. Robert L. McNeill
16437 Wimbledon Lane
Huntington Beach, CA 92649

Mr. David L. Messinger
David L. Messinger & Associates
4009 Webster Street
Oakland, CA 94609

Professor Nathan M. Newmark
Department of Civil Engineering
1211 Civil Engineering Building
University of Illinois
Urbana, IL 61801

Mr. Edward M. O'Connor
256 Ravenna Drive
Long Beach, CA 90803

Mr. Kenward S. Oliphant (Deceased)
657 Howard Street
San Francisco, CA 94105

Mr. Bruce Olsen
1411 Fourth Avenue
Seattle, WA 98101

Mr. Clarkson W. Pinkham
S. B. Barnes & Associates
2236 Beverly Boulevard
Los Angeles, CA 90057

Mr. Robert M. Powell
Perry-Henderson-Powell, Inc.
747 East Green Street, Suite 400
Pasadena, CA 91101

Mr. F. Robert Preece
Testing Engineers, Inc.
300 Montgomery Street
San Francisco, CA 94104

Professor Dixon Rea
3173 Engineering I
University of California
Los Angeles, CA 90024

Dr. John W. Reed
Engineering Decision Analysis Co., Inc.
480 California Avenue, Suite 301
Palo Alto, CA 94306

Mr. Norton S. Remmer
98 Collidge Road
Worcester, MA 01602

Professor F. E. Richart, Jr.
Department of Civil Engineering
University of Michigan
2320 G. G. Brown Lab
Ann Arbor, MI 48105

Professor Jose Roesset
Department of Civil Engineering
Massachusetts Institute of Technology
Cambridge, MA 02139

Mr. Richard C. Rosane
Drummey, Rosane, Anderson
40 William Street
Wellesley, MA 02181

Professor Emilio Rosenblueth
Institute of Engineering
University of Mexico
Mexico 20, D.F., Mexico

Mr. Christ Sanidas
Chief Building Official
Shelby County
160 North Main Street, Room 713
Memphis, TN 38103

Mr. Walter D. Saunders
Martin & Saunders
1820 Wilshire Boulevard
Los Angeles, CA 90057

Dr. John B. Scalzi
Earthquake Engineering
National Science Foundation
1800 G Street N.W.
Washington, DC 20550

Mr. Samuel Schultz
377 South Robertson Boulevard
Beverly Hills, CA 90211

Professor H. Bolton Seed
Department of Civil Engineering
441 Davis Hall
University of California
Berkeley, CA 94720

Mr. Daniel Shapiro
Shapiro, Okino, Hom and Associates
1736 Stockton Street
San Francisco, CA 94133

Mr. Ronald L. Sharpe
Applied Technology Council
480 California Avenue, Suite 205
Palo Alto, CA 94306

Professor Mete A. Sozen
Department of Civil Engineering
3112 Civil Engineering Boulevard
University of Illinois
Urbana, IL 61801

Dr. Richard O. Stone
Department of Geology
University of Southern California
University Park
Los Angeles, CA 90007

Mr. James L. Stratta
Simpson, Stratta & Associates
325 Fifth Street
San Francisco, CA 94107

Dr. Charles Thiel
Division of Advanced Environmental
Research and Technology
National Science Foundation
1800 G Street N.W.
Washington, DC 20550

Professor Mihailo D. Trifunac
University of Southern California
University Park
Los Angeles, CA 90007

Mr. Austin K. Van Dusen
3549 N.E. 16th Street
Seattle, WA 98155

Professor Anestis S. Veletsos
Department of Civil Engineering
Rice University
Houston, TX 77001

Professor Ajit S. Virdee
1717 Daphne Avenue
Sacramento, CA 95825

Mr. G. Robert Voelz
Donald Bentley & Associates
149 New Montgomery
San Francisco, CA 94105

Professor Robert V. Whitman
Department of Civil Engineering
Room 1-253
Massachusetts Institute of Technology
Cambridge, MA 02139

Dr. John H. Wiggins, Jr.
J. H. Wiggins Company
1650 South Pacific Coast Highway
Redondo Beach, CA 90277

Mr. Thomas D. Wosser
H. J. Degenkolb & Associates
350 Sansome Street
San Francisco, CA 94104

Mr. Loring A. Wyllie, Jr.
H. J. Degenkolb & Associates
350 Sansome Street
San Francisco, CA 94104

Mr. George A. Young
Ababian Associates
250 North Nash Street
El Segundo, CA 90245

Mr. Edwin Zacher
615 Creston Road
Berkeley, CA 94708

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

This document contains tentative seismic design provisions for use in the development of seismic code regulations for design and construction of buildings. The provisions represent the result of a concerted effort by a multidisciplinary team of nationally recognized experts in earthquake engineering. Design professionals, researchers, Federal agency representatives, staffs from the model code organizations and representatives from state and local governments from throughout the United States were involved. The provisions are comprehensive in nature and deal with earthquake resistant design of the structural system, architectural and non-structural elements and mechanical-electrical systems in buildings. Both new and existing buildings are included. They embody several new concepts which are significant departures from existing seismic design provisions. An extensive commentary documenting the basis for the provisions is included.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

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