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FEDERAL EMERGENCY MANAGEMENT AGENCY

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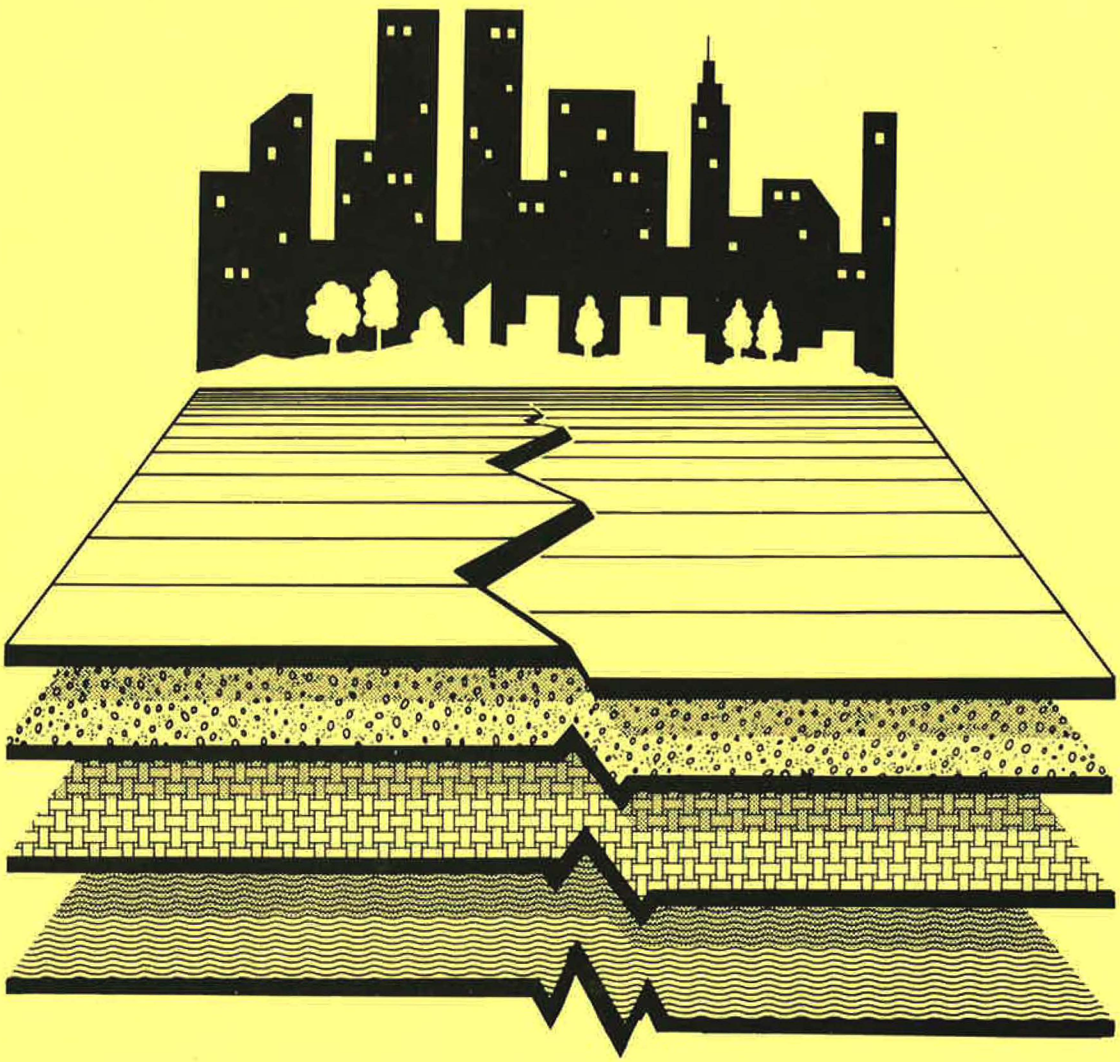
1988 EDITION

Supersedes 1985 Edition

NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings

Part 1
Provisions



EARTHQUAKE HAZARDS REDUCTION SERIES 17

FEDERAL EMERGENCY MANAGEMENT AGENCY

BUILDING SEISMIC SAFETY COUNCIL

The Building Seismic Safety Council (BSSC) is an independent, voluntary body that was established under the auspices of the National Institute of Building Sciences (NIBS) in 1979 as a direct result of nationwide interest in the seismic safety of buildings. Its membership (see inside back cover) represents a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. To fulfill its purpose, the BSSC:

- Promotes the development of seismic safety provisions suitable for use throughout the United States;
- Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;
- Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;
- Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;
- Promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, industry representatives, other members of the building community, and the public;
- Advises government bodies on their programs of research, development, and implementation; and
- Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building-type structures and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. It believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are structured to provide all interested entities (for example, government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community.

The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. The BSSC itself assumes no standards-making and/or -promulgating role; rather, it advocates that standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

BSSC Program on Improved Seismic Safety Provisions

NEHRP
(National Earthquake Hazards Reduction Program)
RECOMMENDED PROVISIONS
FOR THE
DEVELOPMENT OF
SEISMIC REGULATIONS FOR NEW BUILDINGS

1988 Edition

PART 1

PROVISIONS

Prepared by the
Building Seismic Safety Council
for the
Federal Emergency Management Agency

Based on the Applied Technology Council's
1978 *Tentative Provisions for the Development of Seismic Regulations for Buildings*

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
1988

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This report was prepared under Contract EMW-C-0903 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

Building Seismic Safety Council reports include the following:

Abatement of Seismic Hazards to Lifelines: Proceedings of the Building Seismic Safety Council Workshop on Development of an Action Plan, 6 volumes, 1987

Action Plan for the Abatement of Seismic Hazards to New and Existing Lifelines, 1987

Guide to Use of the NEHRP Recommended Provisions in Earthquake-Resistant Design of Buildings, 1985 Edition, 1987

Improving the Seismic Safety of New Buildings: A Community Handbook of Societal Implications (Revised Edition) and *Societal Implications: Selected Readings*, 1986

Improving the Seismic Safety of New Buildings: A Non-Technical Explanation of the NEHRP Recommended Provisions, 1986

NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1988 Edition, 2 volumes, 1988

Seismic Considerations: Elementary and Secondary Schools, 1987

Seismic Considerations: Health Care Facilities, 1987

Seismic Considerations: Hotels and Motels, 1987

Seismic Considerations: Apartment Buildings, 1988

Seismic Considerations: Office Buildings, 1988

For further information concerning any of these documents, contact the Executive Director, Building Seismic Safety Council, 1015 15th St., N.W., Suite 700, Washington, D.C. 20005.

An Action Plan for Reducing Earthquake Hazards of Existing Buildings (1985) and *Proceedings: Workshop on Reducing Seismic Hazards of Existing Buildings* (1985) were developed by the ABE Joint Venture (conducted by the Applied Technology Council, Building Seismic Safety Council, and Earthquake Engineering Research Institute) and are available from FEMA, Earthquake Programs, Washington, D.C. 20472.

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PREFACE

Publication of the 1988 Edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* represents a significant milestone in the continuing effort that FEMA is pleased to sponsor to improve the seismic safety of new buildings in this country. It marks the completion of the first in a planned updating of both the *Provisions* and the complementary publications in this series. As in the case of the original series started in 1985, FEMA encourages widespread dissemination and voluntary utilization of the technology contained in the updated series.

FEMA is very grateful for the unstinting efforts on the part of the large number of volunteer experts and the BSSC Board of Direction and staff who have made possible this updating effort. The survivors of the ill effects of future damaging earthquakes will owe much, perhaps their very lives, to the contributions of these individuals to the seismic safety of buildings.

Federal Emergency Management Agency

INTRODUCTION and ACKNOWLEDGMENTS

The 1988 Edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* is the second edition of the document and, like the 1985 Edition that preceded it, has the consensus approval of the Building Seismic Safety Council membership (see inside back cover). It represents a major product of the Council's multiyear, multitask Program on Improved Seismic Safety Provisions and is intended to continue to serve as a source document for use by any interested members of the building community. (For readers unfamiliar with the program, Appendix C of the *Commentary* volume presents a detailed description of the effort.)

During the effort to update the 1985 Edition for issuance as the 1988 Edition, nine BSSC Technical Committees working under the direction of a Technical Management Committee (TMC) and the BSSC Board of Direction (see the Appendix to the *Provisions* for a list of members of these groups) examined issues left unresolved when the 1985 Edition was published and considered new experience and research data that had become available. Their work resulted in a series of proposals for change to the 1985 Edition that was balloted by the Council membership in February 1988. The results of this ballot highlighted several still controversial issues and prompted the TMC and Board to submit a number of revised proposals to the membership for reballot in June 1988. This 1988 Edition reflects the results of this procedure as well as the efforts and expertise of the many individuals and organizations who have contributed to the development of both editions of the *Provisions* and the work of the Applied Technology Council, which produced the 1978 document on which the *Provisions* was based. A summary of the differences between the 1985 and 1988 Editions of the *Provisions* is presented as Appendix B of the *Commentary* volume.

In presenting this 1988 Edition of the *NEHRP Recommended Provisions*, the BSSC wishes to acknowledge the accomplishments of the many individuals and organizations involved over the years. As Past Chairman of the BSSC Board of Direction Roy G. Johnston noted in the previous edition:

> It is virtually impossible to adequately acknowledge all of the participants in a program of the breadth and depth of the

BSSC program on Improved Seismic Safety Provisions... The earlier work of the Applied Technology Council, the National Bureau of Standards, and the National Science Foundation alone involved hundreds of specialists, all of whom, over many years, gave freely of a great amount of time--time they could have devoted to furthering their own careers or to their families and leisure activities.

The BSSC projects resulting in the 1985 and 1988 Editions of the *Provisions* have involved similar contributions of time and effort as will the projects to develop and publish the 1991 and subsequent triennial editions.

It is difficult to single out a given number or group for special recognition without inadvertently omitting others without whose assistance the update program could not have succeeded; nevertheless, the 1988 Edition of the *Provisions* would not be complete without at least recognizing the following individuals to whom I, acting on behalf of the BSSC Board of Direction, heartily express sincerest appreciation:

- The members of the BSSC Technical Management Committee, especially Chairman Edwin G. Zacher who gave so freely of his time and expertise;
- The members of the nine BSSC Technical Committees; and
- Ugo Morelli, the FEMA Project Officer, whose continuing interest and support have been essential to program continuity.

Appreciation also is due to the BSSC Executive Director James R. Smith and the BSSC staff members and consultants whose talents and experience were crucial to conduct of the program.

At this point I, as Chairman, would like to express my personal gratitude to the members of the BSSC Board of Direction and especially to Past Chairmen William W. Moore and Roy G. Johnston for assuming responsibility for the Council during its formative years and to all those who provided advice, counsel, and encouragement during conduct of the update effort or who otherwise participated in the BSSC program that resulted in the ***NEHRP Recommended Provisions***.

Warner Howe
Chairman, BSSC Board of Direction

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FOR THE DEVELOPMENT OF SEISMIC REGULATIONS
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1988 Edition

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NOTE

Those portions of the 1988 Edition of the *Provisions* that are substantively different from the 1985 Edition are identified in the margins as follows:

Additions or Revisions

Deletions

Not highlighted are editorial changes and the terminology changes required because of the change in Seismic Performance Category designations (from A, B-1, B-2, C, and D to A, B, C, D, and E) and the replacement of the "Seismicity Index" with explicit citations of velocity-related acceleration (A_V).

A summary of the differences between the 1985 and 1988 Editions of the *Provisions* is presented as Appendix B of the *Commentary* volume.

Chapter 1

GENERAL PROVISIONS

1.1 PURPOSE

These provisions present criteria for the design and construction of buildings subject to earthquake ground motions. Their purposes are to minimize the hazard to life for all buildings, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential facilities to function during and after an earthquake. They provide the minimum criteria considered to be prudent and economically justified for the protection of life safety in buildings subject to earthquakes at any location in the United States.

The "design earthquake" ground motion levels specified herein may result in both structural and nonstructural damage, but such damage is expected to be repairable. For ground motions larger than the design levels, the intent of these provisions is that there be a low likelihood of building collapse.

1.2 SCOPE

These provisions establish requirements for the design and construction of new buildings to resist the effects of earthquake motions.

Additions to existing buildings are covered by these provisions as indicated. Existing buildings and alterations and repairs to existing buildings are not covered by these provisions.

EXCEPTION: The following 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 that are located in map areas having a value of A_v less than 0.15.

3. Special structures including, but not limited to, bridges, transmission towers, industrial towers and equipment, piers and wharves, hydraulic structures, and nuclear reactors. These special structures require special consideration of their response characteristics and environment that is beyond the scope of these provisions.

1.3 APPLICATION OF PROVISIONS

New buildings 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 3 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 map areas having a value of A_v equal to or greater than 0.15 need only conform to the requirements for Conventional Light Timber Construction set forth in Sec. 9.7.

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 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 Additions to Existing Buildings

Additions shall be made to existing buildings only as follows:

1.3.2.1 Where an addition is structurally independent from an existing building, it shall be designed in accordance with the provisions of this document for new buildings.

1.3.2.2 Where an addition is not structurally independent from an existing building, such addition may be made under the following conditions: (1) it shall comply with these provisions, (2) it shall not increase the seismic loads to the existing building beyond its lateral force resistance capacity in accordance with the requirements of these provisions, (3) it shall not increase the mass contributing to seismic forces in any elements of the existing building by more than 5 percent, and (4) it shall not decrease the seismic resistance of the existing building.

1.3.2.3 An addition that is not structurally independent from an existing building shall be allowed where the addition and the existing building are made to conform completely with the provisions of this document for new buildings.

1.3.3 Change of Use

When a change in use results in a building being reclassified to a higher Seismic Hazard Exposure Group, such building shall conform to these provisions.

1.3.4 Alterations and Repairs

Alterations and repairs are not covered by these provisions.

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 level of seismicity and the Seismic Hazard Exposure Group are used in assigning buildings to Seismic Performance Categories. Seismic Hazard Exposure Group III is associated with the uses requiring the highest level of protection; Seismic Performance Category E is assigned to provide the highest level of design performance criteria.

1.4.1 Design Ground Motions[1]

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 coefficients A_a and A_v to be used in the application of these provisions shall be determined in accordance with the following procedure.

1.4.1.1 Determine the appropriate Map Areas for the building site from Figures 1-1 and 1-2 and then determine the values for A_a and A_v from either the value on the figure or Table 1-1.

TABLE 1-1
Coefficient A_a and A_v

Map Area from Figure 1-1 (A_a) or 1-2 (A_v)	Value of A_a or A_v
7	0.40
6	0.30
5	0.20
4	0.15
3	0.10
2	0.05
1	0.05

1.4.1.2 Alternatively, values of A_a and A_v may be determined directly from Figures 1-3 and 1-4, respectively; interpolation should be used in reading Figures 1-3 and 1-4.

Alternate Sec. 1.4.1 for Regulatory Agencies That Have Made a Determination of A_a and A_v

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 coefficients A_a and A_v to be used in the application of these provisions are established as:

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[1]For the 1988 Edition of the *Provisions*, alternate maps and an alternate method for establishing design ground motions are presented. See the Appendix to Chapter 1 and the Commentary section on this Appendix.

A_a = ____

and

A_v = ____.

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:

1.4.2.1 Group III

Seismic Hazard Exposure Group III shall be buildings having essential facilities that are necessary for post-earthquake recovery. Also see the requirements for access to and the functionality of essential facilities in Sec. 1.4.2.5 and 1.4.2.6, respectively.

Examples of possible Group III facilities are:

- 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
- Emergency vehicle shelters and garages
- Structures and equipment in emergency preparedness centers
- Structures housing, supporting, or containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released

The Regulatory Agency may designate other buildings where special local conditions make this desirable.

1.4.2.2 Group II

Seismic Hazard Exposure Group II shall be buildings that constitute a substantial public hazard because of occupancy or use.

Examples of possible Group II facilities are:

- Covered structures whose primary occupancy is public assembly with a capacity greater than 300 persons
- Buildings for schools through secondary or day-care centers with a capacity greater than 250 students
- Buildings for colleges or adult education schools with a capacity greater than 500 students

Medical facilities with 50 or more resident incapacitated patients, but not included in Group III
Jails and detention facilities
All structures with an occupancy greater than 5,000 persons
Power generating stations and other public utility facilities not included in Group III and required for continued operation

1.4.2.3 Group I

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

1.4.2.4 Multiple Use

Buildings having multiple uses shall be assigned the classification of the highest Seismic Hazard Exposure Group that occupies 15 percent or more of the total building area.

1.4.2.5 Group III Protected Access

For buildings assigned to Seismic Hazard Exposure Group III, the following apply:

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.

1.4.2.6 Group III Function

Designated Seismic Systems in Seismic Hazard Exposure Group III buildings shall, in so far as practical, be provided with the capacity to function during and after an earthquake.

1.4.3 Seismic Performance Categories

For the purposes of these provisions, all buildings shall be assigned, based on level of the design ground motion coefficient A_v and the Seismic Hazard Exposure Group designated, to a Seismic Performance Category in accordance with Table 1-2.

TABLE 1-2
Seismic Performance Categories

Value of A_v	Seismic Hazard Exposure Group I	II	III
$0.20 \leq A_v$	D	D	E
$0.15 \leq A_v < 0.20$	C	D	D
$0.10 \leq A_v < 0.15$	C	C	C
$0.05 \leq A_v < 0.10$	B	B	C
$A_v < 0.05$	A	A	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 Performance Category E

No building assigned to Category E shall be sited where there is the potential for an active fault to cause rupture of the ground surface at the building.

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.

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 through 12.

1.6.1 Quality Assurance Plan

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

1. Buildings assigned to Category E for the Designated Seismic Systems.

2. Buildings assigned to Categories C and D for the Structural Seismic Resisting Systems.

3. All other buildings determined by the Regulatory Agency.

1.6.1.1 Details of Quality Assurance Plan

The Quality Assurance Plan shall specify the Designated Seismic Systems that 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.

1.6.1.2 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 the system or component. The statement shall clearly show the following:

1. Acknowledgement of awareness of the special requirements contained in the Quality Assurance Plan.

2. Acknowledgement that control will be exercised to obtain conformance with the Design Documents approved by the Regulatory Agency.

3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting, and the distribution of the reports.

4. The person exercising such control and that person's position in the management of the organization.

1.6.2 Special Inspection

The building owner shall employ an approved Special Inspector (who shall be identified as the Owner's Inspector) to observe the construction of all Designated Seismic Systems in accordance with the following requirements:

1.6.2.1 Foundations

Continuous Special Inspection is required during driving of piles, construction of drilled piles, and caisson work.

1.6.2.2 Reinforcing Steel

Special Inspection for reinforcing steel shall be as follows:

1.6.2.2.1 Continuous Special Inspection during the placement of steel in reinforced concrete Special Moment Frames.

1.6.2.2.2 Periodic Special Inspection during the placement of steel in reinforced concrete and reinforced masonry shear walls and Ordinary Moment Frames.

1.6.2.2.3 Continuous Special Inspection during the welding of reinforcing steel.

1.6.2.3 Structural Concrete

Periodic Special Inspection is required during the placement of concrete in drilled piers, caissons, reinforced concrete frames, and shear walls.

1.6.2.4 Prestressed Concrete

Continuous Special Inspection is required during the placement of prestressing steel, during stressing and grouting operations, and during the placement of concrete.

1.6.2.5 Structural Masonry

Special Inspection shall be provided for all structural masonry for Categories D and E:

1. Periodically during the preparation of mortar, the laying of masonry units, and placement of reinforcement;
2. Prior to placement of grout; and
3. Continuously during welding of reinforcement, grouting, consolidation, and reconsolidation.

1.6.2.6 Structural Steel

1.6.2.6.1 Continuous Special Inspection is required for all structural welding.

EXCEPTION: Periodic Special Inspection is permitted for single-pass fillet or resistance welds, provided the welder qualifications and welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the design and plans at the completion of welding.

1.6.2.6.2 Periodic Special Inspection in accordance with Ref. 10.1 or 10.6 for installation and tightening of high-strength bolts is required.

1.6.2.7 Structural Wood

Continuous Special Inspection is required during all field gluing operations. Periodic Special Inspection is required for nailing, bolting, or other fastening.

1.6.2.8 Architectural Components

Special Inspection for Architectural Components designated in Chapter 8 as requiring S or G performance shall be as follows:

1.6.2.8.1 Periodic Special Inspection during the erection and fastening of exterior and interior architectural panels.

1.6.2.8.2 Periodic Special Inspection during the adhesion or anchoring of veneers.

1.6.2.9 Mechanical and Electrical Components

Periodic Special Inspection is required during the 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; and

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.

1.6.3.1 Reinforcing and Prestressing Steel

Special Testing of reinforcing and prestressing steel shall be as follows:

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

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

1.6.3.2 Structural Concrete

Sample at job site and test concrete in accordance with requirements of ACI 318-83. The rate of sampling shall be at least once per day for each class placed.

1.6.3.3 Structural Masonry

Quality Assurance Testing of masonry shall be in accordance with the requirements of Ref. 12.1 (ACI-ASCE 530).

1.6.3.4 Structural Steel

Special Testing of structural steel shall be as follows:

1.6.3.4.1 Welded connections for Special Moment Frames and Eccentrically Braced Frames shall be tested by nondestructive methods conforming to AWS D1.1-85. 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.

1.6.3.4.2 Partial penetration groove welds when used in column splices shall be tested by ultrasonic testing or other approved equivalent methods 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.

1.6.3.4.3 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.

1.6.3.5 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 by actual test on a shaking table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and the forces from Eq. 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 the contractor copies of regular weekly progress reports of his observations, noting therein 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 Systems was constructed substantially in accordance with the Design Documents and applicable workmanship requirements. Work not in compliance shall be noted.

1.6.5 Approved Manufacturer's Certification

Each manufacturer of equipment utilized in a building to be placed in Category E and 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.

APPENDIX TO CHAPTER 1

Alternate Maps and Alternate Method for Establishing Design Ground Motions

This appendix introduces new maps defining the seismic ground-shaking hazard and incorporates a few necessary changes in the expression of certain provisions so that the new maps might appropriately be used with the NEHRP Recommended Provisions. This presentation in an appendix is intended to encourage evaluation of the new maps and new procedures through use. Comment is solicited.

Figures 1-5 through 1-8 are provided for use with this appendix. They provide a different measure of the seismic ground shaking hazard than Figures 1-1 through 1-4 and one of the ground motion parameters is stated in different units. This appendix describes how Figures 1-5 and 1-6 are to be used with the remainder of these provisions. Figures 1-7 and 1-8 are provided to show how A and v vary when the exposure time is increased significantly over that used for Figures 1-1 through 1-6. Figure 1-7 and 1-8 are intended to be advisory only.

Only those sections cited below are changed as noted.

1A.1 Design Ground Motions

Replace Sec. 1.4.1, 1.4.1.1, and 1.4.2 with the following:

1.4.1 Design Ground Motions

The design ground motions are defined in terms of Peak Acceleration and Peak Velocity and are represented by the coefficients A and v, respectively. The coefficients A and v are to be determined as follows:

1. Determine the value of A from Figure 1-5. Interpolation should be used in reading the figure. For use with these provisions, the value of A need not exceed 0.40g.

2. Determine the value of v from Figure 1-6. Interpolation should be used in reading the figure. For use with these provisions, the value of v need not exceed 40 cm/sec.

Alternate Sec. 1.4.1 for Regulatory Agencies That Have Made a Determination of A and v

The design ground motions are defined in terms of Acceleration and Velocity, represented by coefficients A and v respectively. The coefficients A and v to be used in the application of these provisions are established as:

A = _____ and v = _____.

1A.2 Use of A and v with the *Provisions*

The following general rules apply for use of A and v with the *Provisions*:

1. Substitute A directly for A_a in Sec. 4.2.1, Eq. 4-3 and 4-3a, and Sec. 5.5, Eq. 5-3a.

2. Substitute the following coefficients of v for the coefficients of A_v:

   a. 0.012 v for A_v in Sec. 3.7.1; Sec. 8.2.2, Eq. 8-1; and Sec. 8.3.2, Eq. 8-2.

   b. 0.006 v for 0.5 A_v in Sec. 3.7.1, Eq. 3-1, 3-2, and 3-2a.

   c. 0.004 v for $A_v/3$ in Sec. 3.7.5, Sec. 3.7.9, Sec. 7.4.3.

   d. 12 v for 1,000 A_v in Sec. 3.7.6.

   e. 0.014 v for 1.2 A_v in Sec. 4.2.1, Eq. 4-2, and Sec. 5.5, Eq. 5-3.

   f. 0.036 v for 3 A_v in Sec. 5.5, Eq. 5-3b.

   g. v less than 12 for A_v less than 0.015 in Sec. 1.2.

   h. v equal to or greater than 12 for A_v equal to or greater than (=, greater than or equal to, equal or exceeding) in Sec. 1.3.1, Sec. 6A.2.1.2, Sec. 8.3.4, and Sec. 8.3.5.

   i. v = 8, 12, 16 and 25 for A_v = 0.10, 0.15, 0.20 and 0.30, respectively, in Table 6A-1.

   j. v + 16 and 8 for A_v = 0.20 and 0.10, respectively, in Figure 6A-1.

k. Replace Table 1-2 in Sec. 1.4.3 with the following:

TABLE 1A-2
Seismic Performance Categories

	Seismic Hazard Exposure Group		
Value of v	I	II	III
$20 \leq v$	D	D	E
$10 \leq v < 20$	C	D	D
$5 \leq v < 10$	C	C	C
$2.5 \leq v < 5$	B	B	C
$v < 2.5$	A	A	A

l. Replace Table 4-1 in Sec. 4.2 with the following:

TABLE 4A-1
Coefficient for Upper Limit
on Calculated Period

v	C_a
$30 \leq v$	1.2
$20 \leq v < 30$	1.3
$15 \leq v < 20$	1.4
$7.5 \leq v < 15$	1.5
$v < 7.5$	1.7

NOTE: See Sec. 4.2.2 and 5.8 for application of v and C_a.

Chapter 2

DEFINITIONS AND SYMBOLS

2.1 DEFINITIONS

The definitions presented in this section provide the meaning of the terms used in these provisions.

Acceleration

Effective Peak

Coefficient for determining the prescribed seismic forces given in Sec. 1.4.

Effective Peak Velocity-Related

Coefficient for determining the prescribed seismic forces given in Sec. 1.4.

Appendage

An architectural component such as a canopy, marquee, ornamental balcony, or statuary.

Approval

The written acceptance by the Regulatory Agency of documentation that establishes the qualification of a material, system, component, procedure, or person to fulfill the requirements of these provisions for the intended use.

Architectural Equipment

Equipment such as shelving, racks, laboratory equipment, and storage cabinets.

Area Separation Partition

Any partition installed to provide a required fire separation between portions of buildings.

Base

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

Base Shear

Total design lateral force or shear at the base.

Component

A part of an architectural, electrical, mechanical, or structural system.

Code Required

A component required by the Building Code administered by the Regulatory Agency.

Confined Region

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

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

Cross-Tie

A continuous bar having a 135-degree hook with at least a 6-diameter extension at one end and a 90-degree hook with at least a 6-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars.

Design Documents

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

Design Earthquake

The earthquake that produces ground motions at the site under consideration that have a 90 percent probability of not being exceeded in 50 years.

Designated Seismic Systems

The Seismic Resisting System and those architectural, electrical, and mechanical systems and their components that require special performance characteristics.

Diaphragm

A horizontal, or nearly horizontal, system designed to transmit seismic forces to the vertical elements of the Seismic Resisting System.

Frame

Braced

An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a Building Frame or Dual System to resist seismic forces.

Concentrically Braced Frame

A braced frame in which the members are subjected primarily to axial forces.

Eccentrically Braced Frame (EBF)

A diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam-column joint or from another diagonal brace. These short beam segments are called link beams. The following definitions apply:

Diagonal Brace is a member of an EBF placed diagonally in the bay of the frame.

Lateral Support Members are secondary members designed to prevent lateral or torsional bucking of beams in an EBF.

Link Beams are designed to yield in shear and/or bending so that buckling or tension failure of the diagonal brace is prevented. The link beam's length is defined as the clear distance between the diagonal braces or between the diagonal brace and the column face.

Link Beam End Web Stiffeners are vertical web stiffeners placed on the sides of the web at the diagonal brace end(s) of the link beam.

Link Beam Intermediate Web Stiffeners are vertical web stiffeners placed within the link beam.

Link Beam Rotation Angle is defined as the angle between the beam outside of the link beam and the link beam occurring at a total story drift of C_d times the elastic drift at the prescribed design forces. The rotation angle may be computed assuming the EBF bay is deformed as a rigid, ideally plastic mechanism.

Link Beam Shear Yield Strength is defined as the less of V_p of $2M_p/e$, where $V_p = 0.55F_y dt_w$, $M_p = ZF_y$, and e is the link beam length.

Intermediate Moment

A space frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Intermediate Moment Frames of reinforced concrete shall conform to Sec. 11.4.

Ordinary Moment

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, Sec. 11.3, or Sec. 12.3.

Special Moment

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, Sec. 11.5, or Sec. 12.3.

Space Frame

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

Frame System

Building

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

A structural system with an essentially complete Space Frame providing support for vertical loads. A moment resisting frame shall be provided that 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 moment resisting frame together with shear walls or braced frames in proportion to their relative rigidities.

Moment Resisting

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

High Temperature Energy Source

A fluid, gas, or vapor whose temperature exceeds 220 degrees F.

Hoop

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

Inspection, Special

The observation of the work by the Special Inspector to determine compliance with the approved Design Documents and these provisions.

Continuous

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

Periodic

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.

Inspector, Special (who shall be identified as the Owner's Inspector)

A person approved by the Regulatory Agency as being qualified to perform Special Inspection required by the approved Quality Assurance Plan. The quality assurance personnel of a fabricator may be approved by the Regulatory Agency as a Special Inspector.

Joint

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

Laterally Confined

A joint that, 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.

Load

Dead

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

Gravity

W; as defined in Sec. 4.2.

Live

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

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 that shall be equal to either 70 percent of the full snow load or, where conditions warrant and when approved by the Regulatory Agency, not less than 20 percent of the full snow load.

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

P-Delta Effect

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

A detailed written procedure that 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

A system incorporating helical springs, air cushions, rubber-in-shear mounts, fiber-in-shear mounts, or other comparable approved systems.

Stable

A system in which the force displacement ratios are equal in the horizontal and vertical directions.

Restraining Device

A device used to limit the vertical or horizontal movement of the mounting system due to earthquake motions.

Elastic

A fixed restraining device that incorporates an elastic element to reduce the seismic forces transmitted to the structure due to impact from the resilient mounting system.

Fixed

A nonyielding or rigid type of restraining device.

Seismic Activated

An interactive restraining device that is activated by earthquake motion.

Roofing Unit

A unit of roofing material weighing more than 1 pound.

Seismic Forces

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

A classification assigned to a building based on its use as defined in Sec. 1.4.

Seismic Performance Category

A classification assigned to a building as defined in Sec. 1.4.

Seismic Resisting System

That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed herein.

Shear Panel

A floor, roof, or wall component sheathed to act as a shear wall or diaphragm.

Special Lateral Reinforcement

Reinforcement 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.

Story Drift Ratio

The story drift, as determined in Sec. 4.6, divided by the story height.

Story Shear

The summation of design lateral forces at levels above the story under consideration.

Testing Agency

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

The connection of the building's mechanical and electrical distribution systems to the utility or service company's distribution system.

Veneers

Facings or ornamentation of brick, concrete, stone, tile, or similar materials attached to a backing.

Wall

A component, usually placed vertically, used to enclose or divide space.

Bearing

A wall providing support for vertical loads and may be exterior or interior.

Nonbearing

A wall that does not provide support for vertical loads other than its own weight or as permitted by the Building Code administered by the Regulatory Agency. It may be an exterior or interior wall.

Shear

A wall, bearing or nonbearing, designed to resist seismic forces acting in the plane of the wall.

Wall System

Bearing

A structural system with bearing walls providing support for all, or major portions of, the vertical loads. Shear walls or braced frames provide seismic force resistance.

2.2 SYMBOLS

The unit dimensions used with the items covered by the symbols shall be consistent throughout except where specifically noted. The symbols and definitions presented in this section 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.

A_x — The torsional amplification factor.

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_a — Coefficient for upper limit on calculated period; see Table 4-1.

C_c — The seismic coefficient for components of buildings as specified in Tables 8-2 and 8-3 (dimensionless).

C_d — The deflection amplification factor as given in Table 3-2.

C_s — The seismic design coefficient determined in Sec. 4.2 (dimensionless).

$\tilde{C}_s$ — The seismic design coefficient determined in Sec. 6A.2.1 and 6A.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. 4.3.

D_s — The total depth of the stratum in Eq. 6A-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'_m — Ultimate compressive strength of masonry.

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. 5.5.

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. 6A.2 or 6A.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 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. 6A.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. 6A.2.
K_θ	The rocking stiffness of the foundation as defined in Sec. 6A.2.
k	The distribution exponent given in Sec. 4.3.
$\bar{k}$	The stiffness of the building as determined in Sec. 6A.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. 6A.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. 6A.2.3 and 6A.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 — Number of stories, Sec. 4.2.2.

n — Designates the level that is uppermost in the main portion of the building.

P — The performance criteria factor as given in Table 8-1 (dimensionless).

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

P_x — The total unfactored vertical design 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, reduced as permitted in Sec. 2.1.

Q_S — The effect of snow load, reduced as permitted in Sec. 2.1.

R — The response modification coefficient as given in Table 3-2.

r — A characteristic length of the foundation as defined in Sec. 6A.2.1.

r_a — The characteristic foundation length defined by Eq. 6A-8.

r_m — The characteristic foundation length as defined by Eq. 6A-8.

Sec. 2.2

S	The coefficient for the soil profile characteristics of the site as given in Table 3-1.
S_1, S_2, S_3, S_4	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. 6A.2.1 and 6A.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	The total design lateral force or shear at the base.
V_t	The design value of the seismic base shear as determined in Sec. 5.8.
V_x	The seismic design shear in story x 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. 6A.3.
ΔV	The reduction in V as determined in Sec. 6A.2.
ΔV_1	The reduction in V_1 as determined in Sec. 6A.3.
v_s	The average shear wave velocity for the soils beneath the foundation at large strain levels, Sec. 6A.2.
v_{so}	The average shear wave velocity for the soils beneath the foundation at small strain levels, Sec. 6A.2.
W	The total gravity load of the building as defined in Sec. 4.2.
$\bar{W}$	The effective gravity load of the building as defined in Sec. 6A.2 and 6A.3.

$\bar{W}_m$ — The effective modal gravity load determined in accordance with Eq. 5-2.

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

w_i, w_n, w_x — The portion of W that 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. 6A.2.1.

$\tilde{\beta}$ — The fraction of critical damping for the coupled structure-foundation system, determined in Sec. 6A.2.1.

β_o — The foundation damping factor as specified in Sec. 6A.2.1.

γ — The average unit weight of soil.

Δ — The design story drift as determined in Sec. 4.6.1.

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_{max} — The maximum displacement at level x.

δ_{avg} — The average of the displacements at the extreme points of the structure at level x.

δ_x — The deflection of level x at the center of the mass at and above level x, Eq. 4-10.

δ_{xe} — The deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Sec. 4.6.1.

δ_{xem} — The modal deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Sec. 5.6.

δ_{xm}, $\tilde{\delta}_{xm}$ — The modal deflection of level x at the center of the mass at and above level x as determined by Eq. 5-5 and 6A-15.

$\tilde{\delta}_x$, $\tilde{\delta}_{x1}$ — The deflection of level x at the center of the mass at and above level x, Eq. 6A-11 and 6A-14.

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

κ — The overturning moment reduction factor, Eq. 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.

2.3 DEFINITIONS AND SYMBOLS FOR USE WITH THE APPENDIX TO CHAPTER 1

Acceleration

The horizontal ground acceleration from Figure 1-5 expressed as percent of gravity for use as a coefficient for determining the prescribed seismic forces given in Sec. 1.4.

Velocity

The horizontal ground velocity from Figure 1-6 expressed as centimeters per second for use as a coefficient for determining the prescribed seismic forces in Sec. 1.4.

A — Acceleration (%g) determined from Figures 1-5 and 1-7.

v — Velocity (cm/sec) determined from Figures 1-6 and 1-8.

Chapter 3

STRUCTURAL DESIGN REQUIREMENTS

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

A continuous load path, or paths, with adequate strength and stiffness shall be provided to 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.

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 the Soil Profile Types defined below.

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 four types, Soil Profile S_2, Soil Profile S_3, or Soil Profile Type S_4 shall be used depending on whichever Soil Profile Type results in the higher value of seismic coefficient, C_s, as determined in Sec. 4.2.1.

3.2.1.1 Type S_1

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 by other appropriate means of classification, 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.

3.2.1.2 Type S_2

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.

3.2.1.3 Type S_3

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.

3.2.1.4 Type S_4

Soil Profile Type S_4 is a profile with more than 70 feet of soft clays or silts characterized by a shear wave velocity less than 400 feet per second.

3.2.2 Site Coefficient

S is a coefficient for the effects of the site conditions on building response and is given in Table 3-1.

TABLE 3-1
Soil Profile Coefficient

Type	S Factor
S_1	1.0
S_2	1.2
S_3	1.5
S_4	2.0

3.2.3 Soil-Structure Interaction

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

3.3 FRAMING SYSTEMS

As shown in Table 3-2, four types of general framing systems (Bearing Wall, Building Frame, Moment Resisting Frame, and Dual) are recognized for purposes of these provisions. Each type is subdivided by the types of vertical element used to resist lateral seismic forces. For a dual system, a Moment Frame must be provided that is capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Moment Frame and the complementary seismic resisting elements in proportion to their rigidities. Special framing requirements are given in Sec. 3.6 and in Chapters 9 through 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 system types of Table 3-2. The response modification factor, R, and the deflection amplification factor, C_d, are given in Table 3-2 and are used in determining the base shear and the design story drift. Inverted pendulum-type structures associated with buildings are included in Table 3-2.

TABLE 3-2[a]
Response Coefficients

Response Modification Factor R for use in Eq. 4-2, 4-3, 4-3a, 5-3, 5-3a, and 5-3b; Deflection Amplification Factor C_d for use in Eq. 4-10 and 4-11 and in Sec. 10.4.2 and 10.9.3

BEARING WALL SYSTEM

Seismic resisting system	R	C_d
Light framed walls with shear panels	6-1/2	4
Reinforced concrete shear walls	4-1/2	4
Reinforced masonry shear walls	3-1/2	3
Concentrically braced frames	4	3-1/2
Unreinforced masonry shear walls	1-1/4	1-1/4

BUILDING FRAME SYSTEM

Seismic resisting system	R	C_d
Eccentrically braced frames, moment resisting connections at columns away from link	8	4
Eccentrically braced frames, non-moment resisting connections at columns away from link	7	4
Light framed walls with shear panels	7	4-1/2
Concentrically braced frames	5	4-1/2
Reinforced concrete shear walls	5-1/2	5
Reinforced masonry shear walls	4-1/2	4
Unreinforced masonry shear walls	1-1/2	1-1/2

MOMENT RESISTING FRAME SYSTEM

Seismic resisting system	R	C_d
Special moment frames of steel	8	5-1/2
Special moment frames of reinforced concrete	8	5-1/2
Intermediate moment frames of reinforced concrete	4	3-1/2
Ordinary moment frames of steel	4-1/2	4
Ordinary moment frames of reinforced concrete	2	2

[a]Structural and seismic resisting systems listed in this table are defined in Chapter 2. See Sec. 3.3 and 3.6 and Chapters 9 through 12 for special requirements for buildings assigned to various Seismic Performance Categories.

TABLE 3-2 continued

DUAL SYSTEM WITH A SPECIAL MOMENT FRAME CAPABLE OF RESISTING AT LEAST 25 PERCENT OF THE PRESCRIBED SEISMIC FORCES

Complementary seismic resisting elements	R	C_d
Eccentrically braced frames, moment resisting connections at columns away from link	8	4
Eccentrically braced frames, non-moment resisting connections at columns away from link	7	4
Concentrically braced frames	6	5
Reinforced concrete shear walls	8	6-1/2
Reinforced masonry shear walls	6-1/2	5-1/2
Wood sheathed shear panels	8	5

DUAL SYSTEM WITH AN INTERMEDIATE MOMENT FRAME OF REINFORCED CONCRETE OR AN ORDINARY MOMENT FRAME OF STEEL CAPABLE OF RESISTING AT LEAST 25 PERCENT OF THE PRESCRIBED SEISMIC FORCES

Complementary seismic resisting elements	R	C_d
Concentrically braced frames	5	4-1/2
Reinforced concrete shear walls	6	5
Reinforced masonry shear walls	5	4-1/2
Wood sheathed shear panels	7	4-1/2

INVERTED PENDULUM STRUCTURES

Seismic resisting system	R	C_d
Special moment frames of structural steel	2-1/2	2-1/2
Special moment frames of reinforced concrete	2-1/2	2-1/2
Ordinary moment frames of structural steel	1-1/4	1-1/4

3.3.2 Combinations of Framing Systems

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

3.3.2.1 R Value

The value of R in the direction under consideration at any story shall not exceed the lowest value of R obtained from Table 3-2 for the seismic resisting system in the same direction considered above that story.

> 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.

3.3.2.2 Detailing Requirements

For components common to systems having different R values, the detailing requirements required by the higher R value shall be used.

3.3.3 Seismic Performance Categories A, B, and C

Any type of building framing system permitted in these provisions may be used for buildings assigned to Categories A, B, and C except frames limited to Category A only by the requirements of Chapters 11 and 12.

3.3.4 Seismic Performance Category D

Buildings assigned to Category D shall conform to the framing system requirements for Categories A, B, and C and to the additional requirements and limitations of this section.

3.3.4.1 Seismic Resisting Systems

Seismic resisting systems in buildings over 160 feet in height shall be one of the following:

3.3.4.1.1 Moment resisting frame system with Special Moment Frames.

3.3.4.1.2 A Dual System utilizing Special Moment Frames.

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

1. 60 percent when the braced frame or shear walls are arranged only on the perimeter,
2. 40 percent when some of the braced frames or shear walls are arranged on the perimeter,
3. 30 percent for other arrangements.

This system is limited to buildings not over 240 feet in height.

3.3.4.2 Interaction Effects

Moment resisting space frames that are enclosed 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 in Sec. 4.6.

3.3.4.3 Deformational Compatibility

Every structural component not included in the seismic force resisting system in the direction under consideration shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the design story drift Δ as determined in accordance with Sec. 4.6. (See also Sec. 3.8 and Table 3-5.)

3.3.4.4 Special Moment Frames

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

3.3.5 Seismic Performance Category E

The framing systems of buildings assigned to Category E shall conform to the requirements for Category D 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.

3.4 BUILDING CONFIGURATION

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

All structures having irregular features as described in Table 3-3 or Table 3-4 shall be designed to meet the additional requirements of those sections referenced in the tables.

3.4.1 Plan Irregularity

Structures having one or more of the features listed in Table 3-3 shall be designated as having plan irregularity.

3.4.2 Vertical Irregularity

Structures having one or more of the features listed in Table 3-4 shall be designated as having vertical irregularity.

> EXCEPTION: Where no story drift ratio under design lateral load is greater the 1.3 times the story drift ratio of the story above, the structure may be deemed not to have the structural irregularities of Types A or B in Table 3-4. The drift ratio relationship for the top two stories need not be considered. The story drifts for this consideration may be calculated neglecting torsional effects.

3.5 ANALYSIS PROCEDURES

This section prescribes the minimum analysis procedure to be followed. An alternate generally accepted procedure, including the use of an approved site specific spectrum, if desired, may be used in lieu of the minimum applicable procedure. The limitations upon the base shear stated in Chapter 5 apply to any such analysis.

TABLE 3-3
Plan Structural Irregularities

	Irregularity Type and Definition	Reference Section
A	Torsional Irregularity--to be considered when diaphragms are not flexible Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.	3.7.5 4.4.1
B	Re-entrant Corners Plan configurations of a structure and its lateral force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	3.7.5
C	Diaphragm Discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	3.7.5
D	Out-of-Plane Offsets Discontinuities in a lateral force resistance path, such as out-of-plane offsets of the vertical elements.	3.7.5
E	Nonparallel Systems The vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	3.6

TABLE 3-4
Vertical Structural Irregularities

	Irregularity Type and Definition	Reference Section
A	Stiffness Irregularity--Soft Story A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	3.5.3
B	Weight (Mass) Irregularity Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	3.5.3
C	Vertical Geometric Irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force-resisting system in any story is more than 130 percent of that in an adjacent story.	3.5.3
D	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element An in-plane offset of the lateral force-resisting elements greater than the length of those elements.	3.7.5
E	Discontinuity in Capacity--Weak Story A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	3.7.3

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 Categories B and C

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

3.5.3 Seismic Performance Categories D and E

Buildings assigned to Categories D and E shall, as a minimum, be analyzed in accordance with the following procedures:

Table 3.5.3

1. When designated as regular	Chapter 4
2. When designated as irregular and having a height not over 5 stories or 65 feet	Chapter 4
3. When designated as irregular and having a height over 5 stories or 65 feet	Special consideration of dynamic characteristics shall be given
Such buildings having irregularities of Types A, B, or C in Table 3-4	Chapter 5

All buildings designated as irregular shall satisfy the requirements referenced in Tables 3-3 and 3-4.

3.6 DESIGN AND DETAILING REQUIREMENTS

The design and detailing with the requirements of components of structural components of the seismic resisting system ~~and of other structural and nonstructural components shall be as specified in~~ shall comply this section. Foundation design shall conform to the applicable requirements of Chap. 7

3.6.1 Seismic Performance Category A

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

3.6.2 **Seismic Performance Categories B and C**

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

3.6.2.1 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, for Category C, to Sec. 7.4.

3.6.2.2 Materials

3.6.2.1

The materials and the systems composed of those materials shall conform to the requirements and limitations in Chapters 9 through 12 for Categories B and C.

3.6.2.3 Openings

3.6.2.2

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 D**

3.6.4

Buildings assigned to Category D & E shall conform to the requirements for Categories B and C and to the following requirements ~~and limitations~~: 2 of this section.

3.6.3.1 Components

Components of the seismic resisting system and other structural components also shall conform to the requirements of Sec. 3.7.3, 3.7.12, and 7.5.

3.6.3.2 Materials

The materials and the systems composed of these materials shall conform to the requirements and limitations in Chapters 9 through 12 for Category D.

3.6.4 Seismic Performance Category E

Buildings assigned to Category E shall conform to the requirements for Category D and to the following requirements and limitations:

The materials and the systems composed of those materials shall conform to the requirements and limitations of Chapters 9 through 12 for Category E.

3.7 STRUCTURAL COMPONENT LOAD EFFECTS

In addition to the evaluation required by the governing Building Code for other load combinations, 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 Eq. 3-1 or, as applicable, Eq. 3-2 or 3-2a.

$$\text{Combination of load effects} = (1.1 + 0.5\,A_V)\,Q_D + 1.0\,Q_L + 1.0\,Q_S \pm 1.0\,Q_E \quad (3\text{-}1)$$

$$\text{Combination of load effects} = (0.9 - 0.5\,A_V)\,Q_D \pm 1.0\,Q_E \quad (3\text{-}2)$$

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

$$\text{Combination of load effects} = (0.7 - 0.5\,A_V)\,Q_D \pm 1.0\,Q_E \quad (3\text{-}2a)$$

The term $0.5\,A_V$ may be neglected where A_V is equal to 0.05.

3.7.2 Orthogonal Effects

3.6.3.3 3.6.4.1

In buildings assigned to Category B or C, the design seismic forces may be applied separately in each of two orthogonal directions. In buildings assigned to Category D or E, 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 de-

3.6.4.1

signed 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. load structural

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

3.7.3 Discontinuities in Strength of Vertical Resisting System

? The design of buildings assigned to Seismic Performance Category D or E shall consider the potential for 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.

3.6.7.4 Structures assigned to Seismic Performance Category B, C, D or E with a discontinuity in lateral capacity, vertical irregularity Type E as defined in Table 3-4, shall not be over 2 stories or 30 feet in height where the "weak" story has a calculated strength of less than 65 percent of the story above.

EXCEPTION: Where the "weak" story is capable of resisting a total seismic force equal to 0.75 C_d times the design force prescribed in Chapter 4.

3.7.4 Nonredundant Systems

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

3.7.5 Ties and Continuity

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

" 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.

For structures in Seismic Performance Category D or E having a plan irregularity of Type A, B, C or D in Table 3-3 or a vertical irregularity of Type D in Table 3-4, the design forces determined from Chapter 4 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements.

3.7.6 Concrete or Masonry Wall Anchorage

3.6.1.7 Concrete and masonry walls shall be anchored to the roof and all floors that 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 1,000 A_v (lb) 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

3.6.7.6 Collector elements shall be provided that 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

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

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 into the diaphragm, as prescribed in Sec. 3.7.5 or 8.2.2. Diaphragm connections shall be positive mechanical type connections.

3.6.27

3.7.10 Bearing Walls

3.6.28 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 0.1 W_c. Interconnection of dependent wall elements and connections to supporting framing systems shall have sufficient ductility or rotational capacity or 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.

3.6.29

3.7.12 Vertical Seismic Motions for Buildings Assigned to Categories D and E

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

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, $\tilde{\delta}_x$ (as determined in Sec. 6A.2.3), corresponding to the seismic design forces.

The design story drift, Δ, as determined in Sec. 4.6 or 5.8, shall not exceed the allowable story drift Δ_a as obtained from Table 3-5 for any story. For structures with ~~significant torsional deflections,~~ the effect of maximum drift, including torsional effects, shall be considered for stability and damage control.

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TABLE 3-5
Allowable Story Drift Δ_a

	Seismic Hazard Exposure Group		
Buildings	I	II	III
Single story steel buildings without equipment attached to the structural resisting system and without brittle finishes	No Limit	$0.020h_{sx}$	$0.015h_{sx}$
4 stories or less without brittle finishes	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$
All others	$0.015h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Chapter 4

EQUIVALENT LATERAL FORCE PROCEDURE

4.1 GENERAL

The requirements of this chapter provide minimum standards for the seismic analysis of buildings as prescribed in Sec. 3.5.2 and 3.5.3. For purposes of analysis, the building is constructed to be fixed at the base.

4.2 SEISMIC BASE SHEAR

The seismic base shear, V, in a given direction, shall be determined from the following:

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

where

C_s = the seismic design coefficient determined in accordance with Sec. 4.2.1, and

W = the total dead load and applicable portions of other loads listed below:

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater, shall be applicable.

3. Total operating weight of permanent equipment.

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

The value of C_s shall be determined in accordance with Eq. 4-2, 4-3, or 4-3a as appropriate.

4.2.1 Calculation of Seismic Coefficient

When the fundamental period of the building is computed, the seismic design coefficient, C_s, shall be determined from the following:

$$C_s = 1.2\ A_v S/RT^{2/3}, \tag{4-2}$$

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 given in Table 3-1,

R = the response modification factor given in Table 3-2, and

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

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

The soil-structure interaction reduction, when determined in accordance with the Appendix to 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 from the following:

A_v

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

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 or Soil Profile Type S_4 in areas where $A_a \geqslant 0.30$, C_s shall be determined from the following:

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

4.2.2 4.1.2 Period Determination

For use in Eq. 4-2, the fundamental period of the building, T, in the direction under consideration, may be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The period so determined shall not exceed C_aT_a where C_a is given in Table 4-1.

TABLE 4-1
Coefficient for Upper Limit on Calculated Period

A_v	C_a
0.4	1.2
0.3	1.3
0.2	1.4
0.15	1.5
0.1	1.7
0.05	1.7

NOTE: See Sec. 4.2.2 and 5.8 for application of A_v and C_a.

Alternatively, the value of T may be taken equal to the approximate fundamental period of the building, T_a, determined in accordance with Eq. 4-4 or 4-5 as appropriate.

4.2.2.1 4.1.2.1 For buildings in which the lateral force resisting system consists of moment resisting frames capable of resisting 100 percent of the required lateral force and such 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 = 0.030 for concrete frames,

C_T = 0.035 for steel frames, and

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

As an alternate for concrete and steel frame buildings of 12 stories or less with the minimum story height of 10 feet, the formula $T_a = 0.1\ N$, where N = number of stories, may be used in lieu Eq. 4-4.

4.2.2.2 For all other buildings:

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

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

4.3 4.7 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The lateral force, F_x, induced at any level, shall be determined from the following:

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

where

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

w_i and w_x = the portion of W located at or assigned to level i or x;

h_i and h_x = the height above the base to level i or x; and

k = 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.

4.4 HORIZONTAL SHEAR DISTRIBUTION

The seismic design story shear in any story, V_x, shall be determined from the following:

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

The shear, V_x, shall be distributed to the various vertical elements of the seismic resisting system in the story under consideration with due consideration given to the relative stiffnesses of the vertical resisting elements and the diaphragm.

4.4.1 4.3.1 Torsion

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.

For Categories C, D, and E buildings where torsional irregularity exists, as defined in Table 3-4, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following:

$$A_x = (\delta_{max}/1.2\delta_{avg})^2 \tag{4-8}$$

where

δ_{max} = the maximum displacement at level x, and

δ_{avg} = the average of the displacements at the extreme points of the structure at level x.

The value of A_x need not exceed 3.0.

The more severe loading for each element shall be considered for design.

4.5 OVERTURNING

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

The overturning moments shall be determined from the following:

$$M_x = \kappa \sum_{i=x}^{n} F_i \; (h_i - h_x), \tag{4-9}$$

where

κ = 1.0 for the top 10 stories,

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

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 Eq. 4-9 with κ = 0.75 for all building heights.

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 following:

$$\delta_x = C_d \delta_{xe}, \tag{4-10}$$

where

C_d = the deflection amplification factor as given in Table 3-2, and

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).

For determining compliance with the story drift limitation of Sec. 3.8, the deflections δ_x may be calculated as above. For purposes of this drift analysis only, it is permissible to use the computed fundamental period, T, of the building without the upper bound limitation specified in Sec. 4.2.2 when determining drift level seismic design forces.

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 Eq. 4-11, is equal to or less than 0.10:

$$\theta = P_x \Delta / V_x h_{sx} C_d, \qquad (4\text{-}11)$$

where

P_x = the total unfactored vertical design load at story x,

Δ = the design story drift determined as pre scribed in Sec. 4.6.1,

V_x = the seismic design story shear acting in story x,

h_{sx} = the story height of story x, and

C_d = the deflection amplification factor as given in Table 3-2.

When θ is greater than 0.10, the incremental factor related to P-delta effects, a_d, shall be determined by rational analysis (see Part 2, Commentary). The design story drift determined in Sec. 4.6.1 shall be multiplied by the factor ($0.9/1-\theta \geq 1.0$) to obtain the story drift including P-delta effect. ?

The increase in story shears and moments resulting from the increase in story drift shall be added to the corresponding quantities determined without consideration of the P-delta effect.

Chapter 5

MODAL ANALYSIS PROCEDURE

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.

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.

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.

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.

5.5 MODAL BASE SHEAR

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

$$V_m = C_{sm}\bar{W}_m, \qquad (5\text{-}1)$$

where

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

$\bar{W}_m$ = the effective modal gravity load determined as follows:

$$\bar{W}_m = \frac{\left[\sum_{i=1}^{n} w_i\phi_{im}\right]^2}{\sum_{i=1}^{n} w_i\phi_{im}^2} \qquad (5\text{-}2)$$

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:

$$C_{sm} = 1.2\ A_vS/RT_m^{2/3}. \qquad (5\text{-}3)$$

The value of C_{sm} need not exceed 2.5 A_a/R. For Type S_3 or Type S_4 soils in areas where the coefficient $A_a \geqslant 0.3$, C_{sm} need not exceed 2 A_a/R.

EXCEPTIONS:

1. For Soil Profile Type S_3 or Soil Profile Type S_4 soils, C_{sm} for modes other than the fundamental mode that have periods less than 0.3 seconds may be determined as follows:

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

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

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

The reduction due to soil-structure interaction as determined in Sec. 6A.3 may be used.

5.6 MODAL FORCES, DEFLECTIONS, AND DRIFTS

The modal force, F_{xm}, at each level shall be determined as follows:

$$F_{xm} = C_{vxm}V_m \tag{5-4}$$

and

$$C_{vxm} = \frac{w_x\phi_{xm}}{\sum_{i=1}^{n} w_i\phi_{im}}. \tag{5-4a}$$

The modal deflection at each level, δ_{xm}, shall be determined as follows:

$$\delta_{xm} = C_d\delta_{xem}, \tag{5-5}$$

where

$$\delta_{xem} = (g/4\pi^2)(T_m^2F_{xm}/w_x). \tag{5-6}$$

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.

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 Eq. 5-4 and 5-5 shall be computed for each mode by linear static methods.

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, $\bar{V}$, calculated using a period $\bar{T}$ = 1.2 C_aT_a where C_a is as given in Table 4-1. Where V_t is less than $\bar{V}$, the design story shears, moments and drifts, and floor deflections shall be multiplied by $\bar{V}/V_t$. The base shear need not exceed the values determined in accordance with Sec. 4.2.

5.9 HORIZONTAL SHEAR DISTRIBUTION

The provisions of Sec. 4.4 apply.

5.10 FOUNDATION OVERTURNING

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

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

The procedures set forth in the Appendix to Chapter 6 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 procedures 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 the secondary forces associated with the P-delta effect.

APPENDIX TO CHAPTER 6

Soil-Structure Interaction Effects

6A.1 GENERAL

The provisions set forth in this appendix 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 the secondary forces associated with the P-delta effects.

The provisions for use with the Equivalent Lateral Force Procedure are given in Sec. 6A.2 and those for use with the Modal Analysis Procedure are given in Sec. 6A.3.

6A.2 EQUIVALENT LATERAL FORCE PROCEDURE

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

6A.2.1 Base Shear

To account for the effects of soil-structure interaction, the base shear, V, determined from Eq. 4-1 and 4-2 may be reduced to:

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

The reduction, ΔV, shall be computed as follows:

$$\Delta V = [C_s - \tilde{C}_s\,(0.05/\tilde{\beta})^{0.4}]\bar{W}, \tag{6A-2}$$

where

C_s = the seismic design coefficient computed from Eq. 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 Eq. 4-2 using the fundamental natural period of the flexibly supported structure, $\tilde{T}$, defined in Sec. 6A.2.1.1;

$\tilde{\beta}$ = the fraction of critical damping for the structure-foundation system, determined in Sec. 6A.2.1.2; and

$\bar{W}$ = 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.

6A.2.1.1 Effective Building Period

The effective period, $\tilde{T}$, shall be determined as follows:

$$\tilde{T} = T\sqrt{1 + \bar{k}/K_y\,(1 + K_y\bar{h}^2/K_\theta)}, \tag{6A-3}$$

where

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

$$\bar{k} = 4\pi^2(\bar{W}/gT^2); \tag{6A-4}$$

$\bar{h}$ = the effective height of the building which 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; and

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 6A-1 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, and

γ = the average unit weight of the soils.

TABLE 6A-1 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 as follows:

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

where

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

$$\alpha = \bar{W}/\gamma A_o\bar{h}. \qquad (6A-6)$$

r_a and r_m = characteristic foundation lengths, defined by:

$$r_a = \sqrt{A_o/\pi}, \qquad (6A-7)$$

$$r_m = \sqrt[4]{4I_o/\pi}. \qquad (6A-8)$$

where

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.

6A.2.1.2 Effective Damping

The effective damping factor for the structure-foundation system, $\tilde{\beta}$, shall be computed as follows:

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

where

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

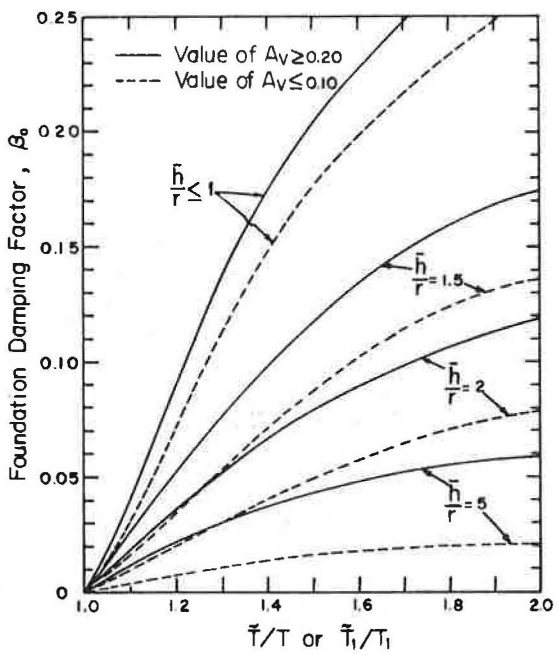

FIGURE 6A-1 Foundation damping factor.

The values of β_o corresponding to A_v = 0.15 in Figure 6A-1 shall be determined by averaging the results obtained from the solid lines and the dashed lines.

The quantity r in Figure 6A-1 is a characteristic foundation length that shall be determined as follows:

For $\bar{h}/L_o \leq 0.5$,

$$r = r_a = \sqrt{A_o/\pi},$$

and for $\bar{h}/L_o \geq 1$,

$$r = r_m = \sqrt[4]{4I_o/\pi},$$

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, and

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

For intermediate values of $\bar{h}/L_o$, 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 Eq. 6-9 shall be replaced by:
>
> $$\beta_o' = (4D_s/V_s\tilde{T})^2 \beta_o, \qquad (6A-10)$$
>
> if $4D_s/v_s\tilde{T} < 1$ where D_s is the total depth of the stratum.

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

6A.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.

6A.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 as follows:

$$\tilde{\delta}_x = \tilde{V}/V\ [M_o h_x/K_\theta + \delta_x], \tag{6A-11}$$

where

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

h_x = the height above the base to the level under consideration, and

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.

6A.3 MODAL ANALYSIS PROCEDURE

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

6A.3.1 Modal Base Shears

To account for the effects of soil-structure interaction, the base shear corresponding to the fundamental mode of vibration, V_1, may be reduced to:

$$\tilde{V} = V_1 - \Delta V_1. \tag{6A-12}$$

The reduction, ΔV_1, shall be computed in accordance with Eq. 6A-2 with $\bar{W}$ taken as equal to the gravity load $\bar{W}_1$ defined by Eq. 5-2; C_s computed from Eq. 5-3 using the fundamental period of the fixed base building, T_1; and $\tilde{C}_s$ computed from Eq. 5-3 using the fundamental period of the elastically supported building T_1.

The period $\tilde{T}_1$ shall be determined from Eq. 6A-3, or from Eq. 6A-5 when applicable, taking $T = \tilde{T}_1$, evaluating $\bar{k}$ from Eq. 6A-4 with $\bar{W} = \bar{W}_1$, and computing $\bar{h}$ as follows:

$$\bar{h} = \frac{\sum_{i=1}^{n} w_i \phi_{i1} h_i}{\sum_{i=1}^{n} w_i \phi_{i1}}; \tag{6A-13}$$

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

6A.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.

The modified modal deflections, $\tilde{\delta}_{xm}$, shall be determined as follows:

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

and

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

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, and

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.

6A.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. 6A.3.2.

Chapter 7

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL

This chapter includes only those foundation requirements that are specifically related to seismic resistant construction. It assumes compliance with all other basic requirements. These requirements include, but are not limited to, provisions for the extent of the foundation investigation, fills to be present or to be placed in the building area, slope stability, subsurface drainage, and settlement control. Also included are pile requirements and capacities and bearing and lateral soil pressure recommendations.

7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS

The resisting capacities of the foundations, subjected to the prescribed seismic forces of Chapters 1 through 6, 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 conform to the requirements of 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 resist loads at acceptable strains considering both the short duration of loading and the dynamic properties of the soil.

7.3 SEISMIC PERFORMANCE CATEGORIES A AND B

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

7.4 SEISMIC PERFORMANCE CATEGORY C

Foundations for buildings classified as Category C shall conform to all of the requirements for Category B construction and to the additional requirements of this section.

7.4.1 Investigation

The Regulatory Agency may require the submission of a written report that 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 slope instability, liquefaction, and 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 reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

7.4.4 Special Pile Requirements

The following special requirements for concrete piles, concrete filled steel pipe piles, drilled piers, or caissons are in addition to all other requirements in the code administered by the Regulatory Agency.

All concrete piles and concrete filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in Chapter 11. The pile cap connection may be made by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the development length is the full development length for compression without reduction in length for excess area. Where special reinforcement at the top of the pile is required, alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided due consideration is given to forcing the hinge to occur in the confined section.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pile, provisions shall be made so that those specified lengths or extents are maintained after pile cut-off.

7.4.4.1 Uncased Concrete Piles

A minimum reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled piles, drilled piers, or caissons in the top one-third of the pile length or a minimum length of 10 feet below the ground. There shall be a minimum of four bars with closed ties (or equivalent spirals) of a minimum 1/4 inch diameter provided at 16-longitudinal-bar-diameter maximum spacing with a maximum spacing of 4 inches in the top 2 feet of the pile. Reinforcement detailing requirements shall be in conformance with Sec. 11.8.2.

7.4.4.2 Metal-Cased Concrete Piles

Reinforcement requirements are the same as for uncased concrete piles.

> EXCEPTION: Spiral welded metal-casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

7.4.4.3 Concrete-Filled Pipe

Minimum reinforcement 0.01 times 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 anchorage into the pile cap.

7.4.4.4 Precast Concrete Piles

Longitudinal reinforcement shall be provided for precast concrete piles with a minimum steel ratio of 0.01. Ties or equivalent spirals shall be provided at a maximum 16-bar-diameter spacing with a maximum spacing of 4 inches in the top 2 feet. Reinforcement shall be full length.

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

7.5 SEISMIC PERFORMANCE CATEGORIES D AND E

Foundations for buildings classified as Categories D and E shall conform to all of the requirements for Category C construction and to the additional requirements of this section.

7.5.1 Investigation

The Regulatory Agency may require the submission of a written report that 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, unless founded directly on rock, as defined in Sec. 3.2.1.1, shall be interconnected by ties. Ties shall conform to Sec. 7.4.3.

7.5.3 Special Pile Requirements

Piling shall be designed to withstand maximum imposed curvatures resulting from seismic forces for free-standing piles in loose granular soils and in Soil Profile Types S_3 and S_4. Piles subject to such deformation shall be designed and detailed in accordance with Sec. 10.7 or Sec. 11.5 for a length equal to 120 percent of the flexural length (point of fixity to pile cap).

7.5.3.1 Uncased Concrete Piles

A minimum reinforcement ratio of 0.005 shall be provided for uncased cast-in-place concrete piles, drilled piers, or caissons in the top one-half of the pile length or a minimum length of 10 feet below ground. There shall be a minimum of four bars with closed ties or equivalent spirals provided at 8-longitudinal-bar-diameter maximum spacing with a maximum spacing of 3 inches in the top 4 feet of the pile. 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.

7.5.3.2 Metal-Cased Concrete Piles

Reinforcement requirements are the same as for uncased concrete piles.

> EXCEPTION: Spiral welded metal-casing of a thickness not less than No. 14 gauge can be considered as providing concrete confinement equivalent to the closed ties or equivalent spirals required in an uncased concrete pile, provided that the metal casing is adequately protected against possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

7.5.3.3 Precast Concrete Piles

Ties in precast concrete piles shall conform to the requirements of Chapter 11 for at least the top half of the pile.

7.5.3.4 Precast-Prestressed Piles

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

7.5.3.5 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 10 percent of the pile compression capacity.

Chapter 8

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS AND SYSTEMS

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 8-2 or 8-3 for performance characteristic level L that are in buildings assigned to Seismic Hazard Exposure Group I which are located in areas with a value of A_v less than 0.15 or that are in buildings assigned to Seismic Hazard Exposure Group II which are located in areas with a a value of A_v less than 0.05 are not subject to the provisions of this chapter.

2. Elevator systems that are in buildings assigned to Seismic Hazard Exposure Group I which are located in areas with a value of A_v less than 0.15 or that are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with a value of A_v less than 0.05 are not subject to the provisions of this chapter.

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 8-3, Footnote *b*.

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 a 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 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 8-1 for use in Eq. 8-1 and 8-2 and Tables 8-2 and 8-3.

TABLE 8-1
Performance Criteria

Performance Characteristic Level	P
Superior (S)	1.5
Good (G)	1.0
Low (L)	0.5

8.2 ARCHITECTURAL DESIGN REQUIREMENTS

8.2.1 General

Systems or components listed in Table 8-2 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 as follows:

$$F_p = A_v C_c P W_c, \tag{8-1}$$

where

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

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

C_c = the seismic coefficient for components of architectural systems as given in Table 8-2 (dimensionless),

P = Performance criteria factor as given in Table 8-1 (dimensionless), and

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

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

> 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.

TABLE 8-2
Seismic Coefficient (C_c) and Performance Characteristic Levels Required for Architectural Systems or Components (see Table 8-1 for S, G, and L designations)

Architectural Components	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels III	II	I
Appendages				
Exterior nonbearing walls	0.9	S	S[b]	S[d]
Wall attachments	3.0	S	S[b]	S[d]
Connector fasteners	6.0			
Veneer attachments	3.0	G	G[a]	L
Roofing units	0.6	G	G[b]	NR
Containers and miscellaneous components (free standing)	1.5	G	G	NR
Partitions				
Stairs and shafts	1.5	S	G[c]	G
Elevator shafts	1.5	S	L[c]	L[e]
Vertical shafts	0.9	S	L[c]	L[f]
Horizontal exits including ceilings	0.9	S	S	G
Public corridors	0.9	S	G	L
Private corridors	0.6	S	L	NR
Full-height area separation partitions	0.9	S	G	G
Full-height other partitions	0.6	S	L	L
Partial-height partitions	0.6	G	L	NR
Structural fireproofing	0.9	S	G[c]	L[f]
Ceilings				
Fire-rated membrane	0.9	S	G[c]	G
Nonfire-rated membrane	0.6	G	G	L
Raised access floors	2.0	S	G	L
Architectural equipment--ceiling, wall, or floor mounted	0.9	S	G	L

TABLE 8-2 Continued

Architectural Components	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels III	II	I
Architectural components--elevator and hoistway structural systems				
Structural frame providing supports for guide rail brackets	1.25	S	G	G
Guide rails and brackets	1.25	S	G	G
Car and counterweight guiding members	1.25	S	G	G

NR = not required.

[a]May be reduced one performance level if the area facing the exterior wall is normally inaccessible for a distance of 10 feet plus 1 foot of each floor of height.

[b]May be reduced one performance level if the area facing the exterior wall is normally inaccessible for a distance of 10 feet and the building is only one story.

[c]Shall be raised one performance level if the building is more than four stories of 40 feet in height.

[d]Shall be raised one performance level if the area facing the exterior wall is normally accessible within a distance of 10 feet plus 1 foot for each floor height.

[e]May be reduced to NR if the building is less than 40 feet in height.

[f]Shall be raised one performance level for an occupancy containing flammable gases, liquids, or dust.

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 accommodate the design story drift as determined in 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 8-2 may provide for a design story drift of $\Delta/2$.

8.2.5 Out-of-Plane Bending

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

8.2.6 Raised Access Floors

Allowance shall be included for the weight of raised floor and equipment to be mounted on the floor.

8.3 MECHANICAL AND ELECTRICAL DESIGN REQUIREMENTS

8.3.1 General

Systems or components listed in Table 8-3 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.

TABLE 8-3
Seismic Coefficient (C_c) and Performance Characteristic Levels Required for Mechanical and Electrical Components (see Table 8-1 for S, G, and L Designations)

Mechanical/Electrical Components[a]	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels		
		III	II	I
Emergency electrical systems (code required) Fire and smoke detection system (code required) Fire suppression systems (code required) Life safety system components	2.00	S	S	S
Elevator machinery and controller anchorage	1.25	S	G	G
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)	0.67	S	G	L
Lighting fixtures	0.67[c]	S	G	L
Ducts and piping distribution systems				
Resiliently supported	2.00	S	G	NR
Rigidly supported	0.67[d]	S	G	NR

TABLE 8-3 Continued

Mechanical/Electrical Components[a]	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels III	II	I
Electrical panelboards and dimmers	0.67	S	G	NR
Conveyor systems (nonpersonnel)	0.67	S	NR	NR

NR = not required.

[a]Where mechanical or electrical components are not specifically listed in Table 8-3, 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.

[b]C_c values listed are for horizontal forces. C_c values for vertical forces shall be taken as one-third of the horizontal values.

[c]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 four times the vertical load.

[d]Seismic restraints may be omitted from the following installations:

(1) Gas piping less than 1 inch inside diameter,
(2) Piping in boiler and mechanical rooms less than 1-1/4 inches inside diameter,
(3) All other piping less than 2-1/2 inches inside diameter,
(4) All electrical conduit less than 2-1/2 inches inside diameter,
(5) All rectangular air-handling ducts less than 6 square feet in cross-sectional area,
(6) All round air-handling ducts less than 28 inches in diameter,
(7) 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, and
(8) 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.

8.3.2 Forces

Mechanical and electrical components and their attachments shall be designed for seismic forces determined as follows:

$$F_p = A_v C_c P a_c a_x W_c, \tag{8-2}$$

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

C_c = the seismic coefficient for components of mechanical or electrical systems as given in Table 8-3 (dimensionless;

a_c = the amplification factor related to the response of a system or component as affected by the type of attachment, determined below; and

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 as follows:

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

where

h_x = the height above the base to level x, and

h_n = the height above the base to level n.

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 $T_c/T > 1.4$	$a_c = 1$
If $T_c/T \geq 0.6$ or ≤ 1.4	a_c = 2 minimum[1]

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[1]See Chapter 8 Commentary.

If mounted on the ground or on a slab in direct contact with the ground	$a_c = 2$

The value of the fundamental period, T, shall be the value used in the design of the building as determined in accordance with Sec. 4.2.2 or 5.4.

The fundamental period of the component and its attachment, T_c, shall be determined as follows:

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

where K is the stiffness of the equipment support attachment determined in terms of load per unit deflection of the center of gravity (lb/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 Eq. 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 Eq. 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 that will not generate forces in excess of those calculated from Eq. 8-2.

8.3.4 Component Design

When the direct attachment method is to be used for components with performance characteristic level S or G in areas with a value of A_v greater than or equal to 0.15, the designer shall require certification from the manufacturer that the components will not sustain damage if subjected to forces equivalent to those resulting from Eq. 8-2.

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

8.3.5.1 Shutoff Devices

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 value of A_v equal or exceeding 0.15 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 that will operate when the ground motion exceeds 0.5 A_a.

8.3.5.2 Utility Connections

Flexible connections for utilities shall be provided for all Seismic Hazard Exposure Groups at the interface of movable portions of the structure to accommodate anticipated displacement.

8.4 ELEVATOR DESIGN REQUIREMENTS

8.4.1 Reference Document

The design and construction of elevators and components shall conform to the requirements of ANSI/ASME A17.1-1987, American National Standard Safety Code for Elevators and Escalators, including Appendix F, Recommended Elevator Safety Requirements for Seismic Risk Zone 3 or Greater, except as modified by provisions of this chapter.

8.4.2 Elevators and Hoistway Structural System

Elevators and hoistway structural systems shall be designed to resist seismic forces in accordance with Eq. 8-1 and Table 8-2.

W_c is defined as follows:

Element = W_c,

Traction Car = C + 0.4 L,

Counterweight = W, and

Hydraulic = C + 0.4 L + 0.25 P,

where

C = the weight of the car,

L = rated capacity,

W = the weight of counterweight, and

P = the weight of plunger.

8.4.3 Elevator Machinery and Controller Anchorage(s)

Elevator machinery and controller anchorages shall be designed to resist seismic forces in accordance with Eq. 8-2 and Table 8-3.

8.4.4 Seismic Controls

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

1. A seismic switch device to provide an electrical alert or command for the safe automatic emergency operation of the elevator system, and

2. A counterweight displacement or derailment device to detect lateral motion of the counterweight.

A continuous signal from (2) or a combination of signals from (1) and (2) will initiate automatic emergency shutdown of the elevator system.

8.4.5 Retainer Plates

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

8.4.6 Deflection Criteria

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

Chapter 9

WOOD

9.1 REFERENCE DOCUMENTS

The quality, testing, design, and construction of members and their fastenings in wood systems that 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 (1986)
Ref. 9.2	American Softwood Lumber Standard	PS 20-70 (1986)
Ref. 9.3	Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber	ASTM D245 (1986)
Ref. 9.4	Methods for Establishing Clear Wood Strength Values	ASTM D2555 (1981)
Ref. 9.5	Softwood Plywood - Construction and Industrial	PS 1-83 (1983)
Ref. 9.6	Mat Formed Wood Particle Board	ANSI A208.1 (1979)
Ref. 9.7	Preservative Treatment by Pressure Process	AWPA C1 (1987), C2 (1987), C3 (1987), C9 (1985), and C29 (1982)
	Pressure Treatment of Timber	ASTM D1760-86a
Ref. 9.8	American National Standard for Wood Products--Structural Glued Laminated Timber	ANSI/AITC A190.1 (1983)

Ref. 9.9	Design and Manufacturing Standard Specification for Structural Glued Laminated Timber of Softwood Species	AITC 117 (1987)
Ref. 9.10	Wood Poles	ANSI 05.1 (1987)
Ref. 9.11	Round Timber Piles	ASTM D25 (1986)
Ref. 9.12	One- and Two-family Dwelling Code	
	International Conference of Building Officials	1983
	Building Officials and Code Administrators	1983
	Southern Building Code Congress	1983
Ref. 9.13	Gypsum Wallboard	ASTM C36-64 (1980)
Ref. 9.14	Fiberboard Nail-Base Sheathing	ASTM D2277-87 (1987)
Ref. 9.15	Plywood Design Specifications	APA (1986)
Ref. 9.16	Diaphragms	APA (1987)

9.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections subjected to seismic forces acting along 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:

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

9.3 SEISMIC PERFORMANCE CATEGORIES A AND B

Buildings assigned to Category A or B 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.

9.4 SEISMIC PERFORMANCE CATEGORY C

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

9.4.1 Detailing Requirements

The construction shall comply with the requirements given below.

9.4.1.1 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.

9.4.1.2 Lag Screws

Washers shall be provided under the heads of lag screws that would otherwise bear on wood.

9.5 SEISMIC PERFORMANCE CATEGORY D

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

9.5.1 Material Limitations

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

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.

9.5.2 Framing Systems

The limitations on framing systems that may be used in Category D are given below.

9.5.2.1 Diaphragms

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

9.5.2.2 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.

9.5.2.3 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. 9.7.3.1 or 9.7.3.2 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 D construction are given below.

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.

9.6 SEISMIC PERFORMANCE CATEGORY E

Buildings assigned to Category E construction shall conform to all of the requirements for Category D 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

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

9.7 CONVENTIONAL LIGHT TIMBER CONSTRUCTION

Wood frame buildings that 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.

9.7.1.1 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.

9.7.1.2 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.

9.7.1.3 Bottom Plates

Studs shall have full bearing on a plate or sill of not less than 2 inch nominal thickness and having a width at least equal to the width of the studs.

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 be not less than 2 inch nominal thickness.

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.

9.7.3 Acceptable Types of Wall Sheathing

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

9.7.3.1 Diagonal Boards

Wood boards of 5/8 inch minimum net thickness applied diagonally on studs spaced not over 24 inches on center.

9.7.3.2 Plywood Panels

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

9.7.3.3 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.

9.7.3.4 Gypsum Sheathing

Gypsum panels not less than 1/2 inch nominal thickness on studs spaced not over 16 inches on center.

9.7.3.5 Particleboard

Particleboard exterior sheathing panels Type 2-M-1 grade, or better, not less than 3/8 inch thick on studs spaced not over 16 inches on center.

9.7.3.6 Gypsum Wallboard

Gypsum wallboard not less than 1/2 inch thick on studs spaced not over 24 inches on center.

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. The shear values may be doubled when identical materials are applied to both sides of the wall.

9.8.2.1 Framing

All framing members used in shear panel construction shall be at least 2 inches in nominal 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.

9.8.2.2 Torsion

Buildings that have one side without shear walls shall meet the following requirements to accommodate 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.

9.8.3.1 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 by 6 nominal boards and three 8d nails for 1 by 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 or 3 inch nominal thickness.

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

9.8.3.2 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 that 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 (plf).

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.

9.8.4.1 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.

9.8.4.2 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.

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 and wood ledgers shall not be used in cross-grain bending or tension.

TABLE 9-1
Allowable Shear in Pounds per Foot (at Working Stress) for Horizontal Plywood Diaphragms with Framing Members of Douglas Fir-Larch or Southern Pine[a] for Seismic Loadings[b]

Panel Grade	Fastener Type	Fastener Minimum Penetration in Framing (in.)	Specified Panel Thicknesses (in.)	Minimum Nominal Width of Framing Member (in.)	Lines of Fasteners	Blocked Diaphragms: Boundaries 6; Other Panel Edges 6	Boundaries 4; Other Panel Edges 6	Boundaries 4; Other Panel Edges 4	Boundaries 2-1/2[d]; Other Panel Edges 4	Boundaries 2-1/2[d]; Other Panel Edges 3	Boundaries 2[d]; Other Panel Edges 3	Boundaries 2[d]; Other Panel Edges 2	Unblocked Diaphragms[e]: Case 1	Unblocked Diaphragms[e]: Cases 2, 3, 4, 5 and 6
						Fastener Spacing (in.) at Diaphragm Boundaries (All Cases), at Continuous Panel Edges Parallel to Load (Cases 3 and 4), and at All Panel Edges[c] — Spacing (in.) per Line at Other Panel Edges							Fastener Spacing at 6 Inches at Supported Edges	
STRUCTURAL I	6d common	1-1/4	5/16	2	1	185	250	---	375	---	420	---	165	125
				3	1	210	280	---	420	---	475	---	185	140
	8d common	1-1/2	3/8	2	1	270	360	---	530	---	600	---	240	180
				3	1	300	400	---	600	---	675	---	265	200
	10d common	1-5/8	15/32	2	1	320	425	---	640	---	730	---	285	215
				3	1	360	480	---	720	---	820	---	320	240
	10d common	1-5/8	23/32	3	2	---	650	870	940	1230	---	---	---	---
				4	2	---	755	980	1080	1410	---	---	---	---
				4	3	---	940	1305	1375	1810	---	---	---	---
	14 gauge staples	2	23/32	3	2	---	600	600	840	900	1040	1200	---	---
				4	3	---	840	900	1140	1350	1440	1800	---	---
C-D, C-C STRUCTURAL II and Other Similar Grades	6d common	1-1/4	5/16	2	1	170	225	---	335	---	380	---	150	110
				3	1	109	250	---	380	---	430	---	170	125
			3/8	2	1	185	250	---	375	---	420	---	165	125
				3	1	210	280	---	420	---	475	---	185	140
	8d common	1-1/2	3/8	2	1	240	320	---	480	---	545	---	215	160
				3	1	270	360	---	540	---	610	---	240	180
			7/16	2	1	255	340	---	505	---	575	---	230	170
				3	1	285	380	---	570	---	645	---	255	190
			15/32	2	1	270	360	---	530	---	600	---	240	180
				3	1	300	400	---	600	---	675	---	265	200

C-D, C-C STRUCTURAL II and Other Similar Grades	10d common	1-5/8	15/32	2	1	290	385	---	575	---	655	---	255	190
				3	1	325	430	---	650	---	735	---	290	215
			19/32	2	1	320	425	---	640	---	730	---	285	215
				3	1	360	480	---	720	---	820	---	320	240
			23/32	3	2	---	645	870	935	1225	---	---	---	---
				4	2	---	750	980	1075	1395	---	---	---	---
				4	3	---	935	1305	1390	1510	---	---	---	---
	14 gauge staples	2	23/32	3	2	---	600	600	820	900	1020	1200	---	---
				4	3	---	820	900	1120	1350	1400	1510	---	---

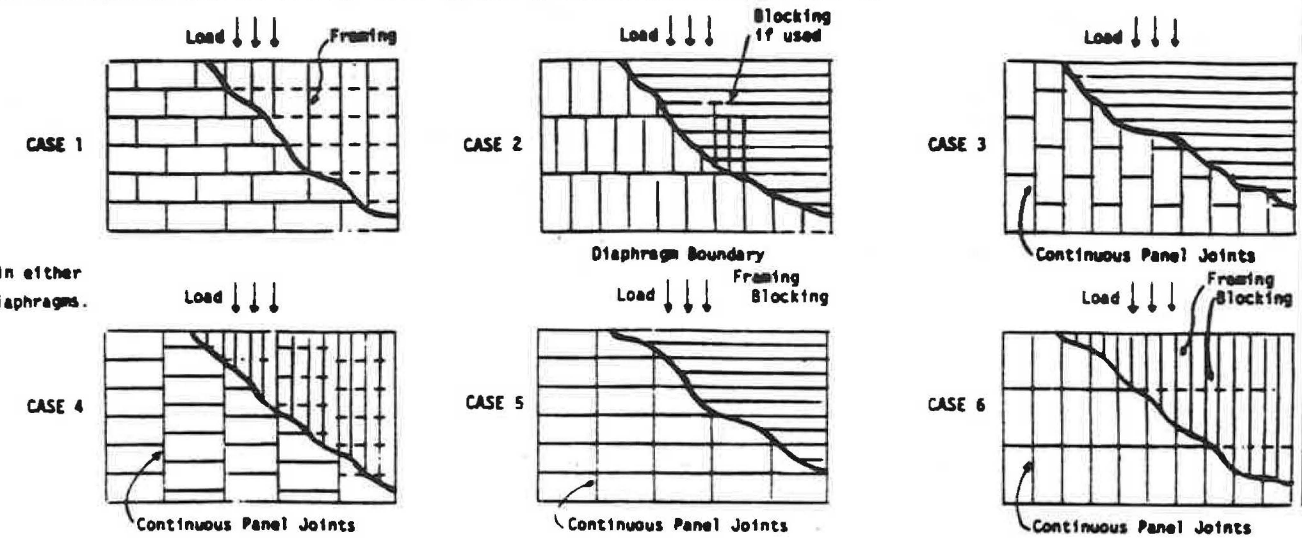
[a]Allowable shear values for fasteners in framing members of other species set forth in Table 8.1A of Ref. 9.1 shall be calculated for all grades by multiplying the values for fasteners in STRUCTURAL I by the following factors: Group III, 0.82, and Group IV, 0.65.

[b]Space nails along intermediate framing members at 10 inch centers for floors and 12 inch centers for roofs except where spans are greater than 32 inches, space nails at 6 inch centers.

[c]Maximum shear for Cases 3, 4, 5, and 6 is limited to 1200 pounds per foot.

[d]For values listed for 2 inch nominal framing member width, the framing members at adjoining panel edges shall be 3 inch nominal width. Nails at panel edges shall be placed in two lines at these locations.

[e]Blocked values may be used for 1-1/8 inch panels with tongue-and-groove edges where 1 inch by 3/8 inch crown by No. 16 gauge staples are driven through the tongue-and-groove edges 3/8 inch from the panel edge so as to penetrate the tongue. Staples shall be spaced at one half the boundary nail spacing for Cases 1 and 2 and at one third the boundary nail spacing for Cases 3 through 6.



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

Table 9-2

TABLE 9-2
Allowable Shear for Wind or Seismic Forces in Pounds per Foot for Plywood Shear Walls with Framing of Douglas Fir-Larch or Southern Pine[a]

Plywood Grade	Nail Size	Penetration in Framing (inches)	Plywood Thickness (inches)	Plywood Applied Direct to Framing[b]				Nail Size	Plywood Applied Over 1/2-inch Gypsum Sheathing[b]			
				6	4	3	2[c]		6	4	3	2[c]
Structural I	6d[d]	1-1/4	5/16	200	300	390	510	8d[d]	200	300	390	510
	8d[d]	1-1/2	3/8	230[e]	360[e]	460[e]	610[e]	10d[d]	280	430	550[c]	730
	8d[d]	1-1/2	15/32	280	430	550	730	10d[d]	280	430	550[c]	730
	10d[d]	1-5/8	15/32	340	510	665[c]	870	-	-	-	-	-
C-D, C-C Structural II and Other Grades Covered in PS 1-83	6d[d]	1-1/4	5/16	180	270	350	450	8d[d]	180	270	350	450
	6d[d]	1-1/4	3/8	200	300	390	510	8d[d]	200	300	390	510
	8d[d]	1-1/2	3/8	220[e]	320[e]	410[e]	530[e]	10d[d]	260	380	490[c]	640
	8d[d]	1-1/2	15/32	260	380	490	640	10d[d]	260	380	490[c]	640
	10d[d]	1-5/8	15/32	310	460	600[c]	770	-	-	-	-	-
	10d[d]	1-5/8	19/32	340	510	665[c]	870	-	-	-	-	-
Plywood Panel Siding in Grades Covered in PS 1-83	6d[f]	1-1/4	5/16	140	210	275	360	8d[f]	140	210	275	360
	8d[f]	1-1/2	3/8	130[e]	200[e]	260[e]	340[e]	10d[f]	160	240	310[c]	410

[a]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 Ref. 9.1, Table 8.1A shall be calculated for all grades by multiplying the values for common and galvanized nails in Structural I and galvanized casing nails in other grades by the following factors: Group III, 0.82, and Group IV, 0.65.

[b]Nail spacing at plywood panel edges.

TABLE 9-2 Continued

[c]Framing shall be 3-inch nominal or wider and nails shall be staggered where nails are spaced 2 inches on center and where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches on center.

[d]Common or galvanized box nails.

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

[f]Galvanized casing nails.

Table 9-3

TABLE 9-3
Allowable Working Stress Shears for Wind or Seismic Loading on Vertical Shear Panels of Fiberboard Sheathing Board[a]

Size/Application	Nail Size	Shear Value 3-Inch Nail Spacing Around Perimeter and 6-Inch at Intermediate Points (plf)
1/2 in. by 4 ft by 8 ft	No. 11 ga. galv. roofing nail 1-1/2 in. long, 7/16 in. head	125[b]
25/32 in. by 4 ft by 8 ft	No. 11 ga. galv. roofing nail 1-3/4 in. long, 7/16 in. head	175

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

[b]The shear value may be 175 plf for 1/2 inch by 4 foot by 8 foot fiberboard classified as nail-based sheathing.

TABLE 9-4
Allowable Working Stress Shears for Shear Walls of Lath and Plaster, Gypsum Sheathing Board, and Gypsum Wallboard Wood-Framed Assemblies[a]

Type of Material		Thickness of Material	Wall Construction	Nail Spacing Maximum[b]	Shear Value (psf)	Minimum Nail Size
Woven or welded wire lath and portland cement plaster		7/8 in.	Unblocked	6 in.	180	No. 11 ga. 1-1/2 in. long, 7/16 in. diam. head, or No. 16 ga. staples having 7/8 in. long legs
Gypsum lath, plain or perforated		3/8 in. lath and 1/2 in. plaster	Unblocked	5 in.	100	No. 13 ga. 1-1/8 in. long, 19/64 in. head, plasterboard blued nail
Gypsum sheathing board	2 ft x 8 ft	1/2 in.	Unblocked	4 in.	75	No. 11 ga. 1-3/4 in. long, 7/16 in. head, diamond point, galvanized
	4 ft x 8 ft	1/2 in.	Blocked	7 in.	175	
	4 ft x 8 ft	1/2 in.	Unblocked	4 in.	100	
Gypsum wallboard		1/2 in.	Unblocked	7 in.	100	5d cooler nails
		1/2 in.	Unblocked	4 in.	125	
		1/2 in.	Blocked	7 in.	125	
		1/2 in.	Blocked	4 in.	150	
		5/8 in.	Blocked	4 in.	175	6d cooler nails
		5/8 in.	Blocked	Base ply 9 in.	250	Base ply--6d cooler nails
		5/8 in.	Two ply	Face ply 7 in.	250	Face ply--8d cooler nails

[a]Shear walls shall not be used to resist loads imposed by masonry or concrete walls.

[b]Applies to nailing at all studs, top and bottom plates, and blocking.

Chapter 10

STEEL

10.1 REFERENCE DOCUMENTS

The quality and testing of steel materials and the design and construction of steel components that 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, November 1, 1978, including Supplement No. 1, Effective March 11, 1986

Ref. 10.2 Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute (AISI), August 10, 1986

Ref. 10.3 The Specifications for the Design of Cold-formed Stainless Steel Structural Members, AISI, 1974 Edition

Ref. 10.4 Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders, Steel Joist Institute, 1986 Edition

Ref. 10.5 The Criteria for Structural Applications for Steel Cables for Buildings, AISI, 1973 Edition.

Ref. 10.6 Load and Resistance Factor Design Specification for Structural Steel Buildings, American Institute of Steel Construction, September 1, 1986

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.

Sec. 10.2/Sec. 10.2.1.2

The value of ϕ shall be as follows:

Members, connections, and base plates that develop the strength of the members or structural systems	$\phi = 0.90$
Connections that do not develop the strength of the member or structural system, including connection of base plates and anchor bolts, or do not conform to Sec. 10.7.6	$\phi = 0.67$
Metal deck diaphragms	$\phi = 0.60$
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:

10.2.1.1 Load Combination

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."

10.2.1.2 Euler Stress

Sec. 1.6.1. The definition of F_e' for the purpose of determining the strength of structural steel members shall read as follows:

$$F_e' = \frac{\pi^2 E}{(K l_b / r_b)^2}$$

where

l_b = the actual length in the plane of bending,

r_b = the corresponding radius of gyration, and

K = the effective length factor in the plane of bending.

10.2.1.3 Member Strength

Amend first paragraph of 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."

10.2.1.4 Shear Strength

In Sec. 2.5, substitute 0.60 for 0.55 in Formula 2.5-1 and delete the word "factored" from the definition of V_u.

10.2.1.5 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 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:

10.2.2.1 Member Strength

10.2.2.1.1 Modify Sec. A4.4 of Ref. 10.2 by substituting 0.60 for the multiplier of 0.75.

10.2.2.1.2 Modify Sec. 3.9.1 and the first paragraph of Sec. 3.9.2 of Ref. 10.3 by substituting 70 percent for the 33-1/3 percent increase to determine the strength of the cold-formed members subjected to seismic forces alone or seismic forces in combination with dead and live loads.

10.2.2.2 Effective Width

Add the following to 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. 3.9.1 or 3.9.2."

10.2.2.3 Steel Deck Diaphragms

Steel deck diaphragms made from materials conforming to the requirements of Ref. 10.2 and 10.3 may be assigned strength values in accordance with one of the following:

1. The tested strength values as approved by the Regulatory Agency,

2. The tested strength values defined as the mean minus two times the standard deviation of at least three tests, or

3. Two times the published allowable working stress values as approved by the Regulatory Agency.

Installation, including fasteners, shall be in conformance with the procedures used for the tests establishing the strengths.

10.2.3 Steel Cables

Reference 10.5, 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 factor of 1.1 shall be applied to the prestress force to be added to the load combination of Sec. 3.1.2 of Ref. 10.6.

10.3 SEISMIC PERFORMANCE CATEGORIES A AND B

Buildings assigned to Categories A or B may be of any type of steel construction permitted in the reference documents.

10.4 SEISMIC PERFORMANCE CATEGORY C

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

10.4.1 Space Frames

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

10.4.2 Braced Frames

Braced frames used with or without accompanying moment frames shall be designed and constructed in accordance with Ref. 10.1 or Ref. 10.2 or Ref. 10.3. The brace connections shall be designed to develop the tensile yield capacity of the brace or to provide tensile deformation equivalent to C_d (Table 3-2) times the brace deformation caused by the seismic design forces. For concentrically braced frames in buildings over two stories, the sum of the strengths of the tension and compression bracing in one horizontal direction shall be substantially equal to the sum of the strengths of their bracing in the opposite horizontal direction.

10.5 SEISMIC PERFORMANCE CATEGORY D

10.5.1 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. 10.7.

> EXCEPTION: Ordinary Moment Frames may be used in one- and two-story buildings.

10.5.2 Braced Frames

Where a concentric braced frame system is used alone or in combination with ordinary moment frames or special moment frames as the seismic resisting system, the concentric braced frames shall conform to the requirements of Sec. 10.8. Where an eccentric braced frame system (EBF) is used as the seismic resisting system, it shall conform to the requirements of Sec. 10.9.

10.6 SEISMIC PERFORMANCE CATEGORY E

10.6.1 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. 10.7.

> EXCEPTION: Ordinary Moment Frames may be used in one-story buildings.

10.6.2 Braced Frames

Concentric braced frame systems shall be used only as part of a dual system. Where a concentric braced frame system is used in combination with ordinary moment frames or special moment frames as the seismic resisting system, the concentric braced frames shall conform to the requirements of Sec. 10.8. Where an eccentric braced frame (EBF) system is used as the seismic resisting system, it shall conform to the requirements of Sec. 10.9.

10.7 SPECIAL MOMENT FRAME REQUIREMENTS

Special Moment Frames shall be designed in accordance with Part 2 of Ref. 10.1 with the following modifications:

10.7.1 Substitute the following for the last three paragraphs of 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 such 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 flexural strength of flexural members shall be determined by the moment $M_p = ZF_y$."

10.7.2 Substitute the following for Sec. 2.2:

"Structural steel shall conform to one of the following ASTM specifications, A36/36M-84a, A441-84, A500-84, A501-84, A572-84; (Grades 42 and 50), or A588-84a.

"EXCEPTION: Structural Steel ASTM A283-75 Grade D may be used for base plates."

10.7.3 Sec. 2.3.1 shall not apply and the last sentence of Sec. 2.3.2 shall be modified to read:

"The axial force in the columns shall not exceed 0.6 P_y."

10.7.4 Add the following to 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 end of the column with the yield strengths 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 axial tension load as specified in Eq.3-2a."

10.7.5 Add the following to Sec. 2.5:

"Shear in frame beams and columns and their connections shall be determined on the basis of dead and live loads acting in conjunction with moments equal to the member flexural capacities at critical sections. However, these shears shall not be 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."

Applicable definitions of terms remain unchanged.

10.7.6 Add the following to Sec. 2.8:

"Each girder moment connection to a column shall be capable of developing in the girder the full bending strength of the girder unless it can be shown that adequate rotation can be obtained by deformation of the connected materials. The connection consists of only those elements that connect the member to the joint. A girder-to-column connection may be considered to develop the bending strength of the girder if the girder flanges have a complete butt weld connection to the column and the girder web is connected to the column through a welded connection or a high strength bolted connection, provided the high-strength bolts are installed to the minimum tension for a slip-critical connection."

10.7.7 Change the start of the second paragraph in 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...."

10.8 CONCENTRICALLY BRACED FRAME REQUIREMENTS

The requirements of this section apply to concentric bracing systems, including bracing used in combination with moment resisting frame systems, whether or not the system is a dual system. This section applies to all bracing systems except eccentrically braced frames (EBF) designed in accordance with Sec. 10.9. When bracing is provided in moment resisting frame systems to control drift, as permitted by Sec. 3.3.4.2, the beams and columns incorporating the bracing shall be designed to carry the brace forces in accordance with this section in addition to the appropriate forces from gravity and seismic loads determined from their performance as moment resisting frames. For Categories D and E, bracing elements shall not terminate within the clear span of members whose main function is to carry axial gravity loads (columns).

10.8.1 Bracing Members

10.8.1.1 Member Strength

The strength of the member in tension is equal to its tensile yield strength and the strength of the member in compression is equal to its buckling strength. These strengths are determined by Ref. 10.1, Part 2, or by Ref. 10.6 provisions. The effective length of the bracing member in compression should be estimated on a rational basis.

10.8.1.2 Lateral Force Distribution

The lateral force capacity along any line of bracing shall not exceed the capacity in the reverse direction by more than 10 percent. For the purpose of this determination, a line of bracing is defined as a single line or as parallel lines within 10 percent of the perpendicular dimension of the building provided that the floor diaphragm is capable of transmitting the forces between parallel lines.

10.8.1.3 Width-Thickness Ratio

The width-thickness ratio of stiffened and unstiffened compression elements used as braces shall not exceed the values given in Ref. 10.1, Part 2, or Ref. 10.6, Table C-B5.1. For rectangular tube sections, the width-thickness ratio shall not exceed $90/\sqrt{F_y}$ unless the tube walls are stiffened by other means.

10.8.1.4 Built-up Members

The L/r of the individual parts of built-up members between stitches calculated about their own axes shall not be greater than 75 percent of the governing slenderness ratio of the member as a whole.

10.8.2 Beams

Beams with bracing members connected at locations not at the beam ends shall be designed for gravity forces for the full span length. Brace support of gravity loads shall be considered in the design of the bracing members, but not for the design of the beams.

10.8.3 Columns

The design forces for the columns consist of those from the gravity loads plus the maximum forces that the braces can deliver to the columns taking into consideration the bracing yield and buckling strengths and the moment capabilities of the associated beams.

10.8.4 Bracing Member Connections

In Seismic Performance Categories D and E, connections shall be designed to develop the full tensile yield capacity of the member.

10.8.4.1 Net Area

In bolted brace connections, the ratio of the minimum effective net section area to the gross section area shall not be less than 1.2 times the ratio of the material minimum yield strength to the minimum tensile strength.

10.8.4.2 Stitches

For a brace that will buckle out-of-plane, the first stitch on each side of the midlength of a built-up member shall be designed to transmit a force equal to 50 percent of the yield capacity of one element to the adjacent element. Bolted stitches shall not be placed at the midlength of a brace member.

10.8.4.3 Gusset Plates

The end gusset plates shall be designed to carry the full axial load and end moment capacities of the bracing member for in-plane buckling. For out-of-plane buckling, the gusset plate shall have a clear end length of two times the gusset plate thickness and shall be able to carry the full compression capacity of the brace member without local buckling of the gusset plate. The bolts or welds shall be designed to transmit the brace forces along the centroids of the brace elements. The length should be sufficient to avoid tearing failure.

10.8.5 Bolts

Bolts used in seismic resistant connections shall be fully tightened.

10.9 ECCENTRICALLY BRACED FRAME REQUIREMENTS

Eccentrically braced frames shall be designed so that under severe earthquake loading yielding occurs primarily in the link beam while all other frame members remain essentially elastic. EBFs shall be designed in accordance with the following provisions:

10.9.1 Link beams shall satisfy compact section requirements of Ref. 10.6, Sec. B5 and Table C-B5.1 for seismic applications. The nominal yield strength of steel used for link beams shall not exceed F_y = 50 ksi.

10.9.2 The shear force in a link beam produced by the prescribed design forces shall not exceed the link beam shear yield strength. The web of a link beam shall be single thickness without doubler plate reinforcement and without openings. Link beam shear yield strength is the lesser of V_p or $2M_p/e$, where $V_p = 0.55F_y dt_w$, $M_p = ZF_y$, and e is the link beam length.

10.9.3 The link beam rotation angle is the angle between the beam outside the link and the link beam occurring at a total story drift of C_d times the elastic drift at the prescribed design forces and may be computed assuming the EBF bay is deformed as a rigid--ideally plastic--mechanism.

10.9.3.1 This rotation angle shall not exceed (except as noted in Sec. 10.9.10.3 below) 0.08 radian for link beams of length 1.6 M_p/V_p or less.

10.9.3.2 This rotation angle shall not exceed (except as noted in Sec. 10.9.10.3 below) 0.02 radian for link beams of length 2.6 M_p/V_p or greater.

10.9.3.3 Linear interpolation shall be used for link beams of lengths between 1.6 M_p/V_p and 2.6 M_p/V_p.

10.9.4 For link beam lengths 1.6 M_p/V_p or greater, the combined flexural strength of a brace and a continuous beam shall exceed M_p where the diagonal brace is rigidly joined to the continuous beam. The strength of these members shall be determined using interaction equations for axial force and bending moment.

10.9.5 If the axial force in a link beam at the prescribed design forces is less than 0.15 P_y, where $P_y = AF_y$, the effect of axial force on link strength and plastic rotation capacity may be neglected. Where axial force in the link exceeds 0.15 P_y, the axial force and flexural moment shall be transmitted by the link beam flanges only.

10.9.6 Full depth web stiffeners shall be provided on both sides of the beam web at the diagonal brace ends of the link beam. These stiffeners shall have a combined width not less than (b_f-2t_w) and a thickness not less than the larger of 0.75 t_w or 3/8 inches.

10.9.7 Link beams shall be provided with intermediate web stiffeners as follows:

10.9.7.1 Link beams of length 1.6 M_p/V_p or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding 38 t_w-d/5 for rotation angle of 0.08 radian or 56 t_w-d/5 for rotation angles of 0.04 radian or less. Interpolation shall be used for values between 0.04 and 0.08.

10.9.7.2 Link beams of length greater than 2.6 M_p/V_p and less than 5 M_p/V_p shall be provided with intermediate web stiffeners placed at a distance of b_f from each end of the link where b_f is the link beam flange width.

10.9.7.3 Link beams of length between 1.6 M_p/V_p and 2.6 M_p/V_p shall be provided with intermediate web stiffeners meeting the requirements of both Sec. 10.9.7.1 and Sec. 10.9.7.2 above.

10.9.7.4 No intermediate web stiffeners are required in link beams of length greater than 5 M_p/V_p.

10.9.8 Intermediate link beam web stiffeners shall be full depth. For beams less than 24 inches in depth, stiffeners are required only on one side of the beam web. The thickness of one-sided stiffeners shall not be less than the larger of t_w or 3/8 inch, and the width shall not be less than $(b_f/2)-t_w$. For beams 24 inches in depth or greater, intermediate stiffeners are required on both sides of the web.

10.9.9 Fillet welds connecting the link beam web stiffeners to the beam web shall develop a force of at least $A_{st}F_y$, and those connecting the stiffener to the flanges shall develop a force of at least $A_{st}F_y/4$, where A_{st} = bt of stiffener, b = width of stiffener plate, and t = thickness of stiffener plate.

10.9.10 When a link beam is connected to a column, the following requirements shall be met:

10.9.10.1 Link beams adjacent to columns shall not exceed the length of 1.6 M_p/V_p unless the beam column connection is reinforced.

10.9.10.2 The link beam flanges shall have full penetration welds to the column. The connection of the link beam web to the column shall be welded to develop the shear strength of the link beam.

10.9.10.3 Where the link beam is connected to the column web, the link beam flanges shall have full penetration welds to the connection plates and the web connection shall be welded to develop the shear strength of the link beam. The rotation angle of the link beam shall not exceed 0.020 radian for any link beam length when the link beam is connected to the column web.

10.9.11 Lateral supports of link beam ends shall have a minimum compressive strength of 4 percent of the beam flange yield strength computed as $F_y b_f t_f$.

10.9.12 The strength of each diagonal brace shall be adequate to resist the forces generated by at least 1.5 times the shear yield strength of the link beam.

10.9.13 Diagonal brace to beam connections shall develop the strength of the diagonal brace and transfer this force to the beam web. No part of the diagonal brace to link beam connection shall extend over the link beam length.

10.9.14 Columns shall be designed to remain elastic at the yield capacity of the EBF bay except that F_y for beams shall be increased by a factor of 1.25.

10.9.15 The beam outside the link beam shall be provided with sufficient lateral support to maintain the stability of the beam under the forces generated by at least 1.5 times the shear yield strength of the link beam. Lateral supports shall be provided at both the top and bottom flanges of the beam and shall have a strength to resist at least 1.5 percent of the beam flange yield strength computed as $F_y b_f t_w$.

10.9.16 Beam to column connections may be designed as partially restrained (simple) connections. The connection shall have a strength to resist rotation about the longitudinal axis of the beam based on two equal and opposite rotation forces of at least 0.01 $F_y b_f t_f$ acting laterally on the beam flanges.

Appendix to Chapter 10

LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

As an alternative to the allowable stress provisions of Ref. 10.1 as modified by Chapter 10, steel structures may be designed in accordance with Ref. 10.6. When Ref. 10.6 is used, Sec. 10.7 shall be replaced by the following:

10A.7 SPECIAL MOMENT FRAME REQUIREMENTS

Special Moment Frames shall be designed in accordance with Ref. 10.6 (AISC, 1986), including Eq. C-F1.1 and the requirements of Table C-B5.1 for seismic conditions, with the following modifications:

10A.7.1 Add the following to Sec. A2.2:

"Special Moment Frames shall satisfy the requirements for Type FR construction in the plane of the frame as provided in Sec. A2.2 Type PR construction is permitted for members between such frames. Connections joining a portion of a structure designed on the basis of plastic design provisions 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 Ref. 10.6 shall govern.

"The flexural strength of flexural members shall be determined by the moment $M_p = ZF_y$."

10A.7.2 Substitute the following for Sec. A3.1:

"Structural steel shall conform to one of the following ASTM specifications, A36/36M-84a, A441-84, A500-84, A501-84, A572-84; (Grades 42 and 50), or A588-84a.

"EXCEPTION: Structural Steel ASTM A283-75 Grade D may be used for base plates."

10A.7.3 Sec. C2.1 of Ref. 10.6 shall not apply and the last sentence of Sec. C2.2 of Ref. 10.6 shall be modified to read:

"The axial force in the columns shall not exceed 0.6 P_y."

10A.7.4 Add the following to Sec. E2:

"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 end of the column with the yield strengths 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 axial tension load as specified in Eq. 3-2a."

10A.7.5 Add the following to Sec. F2.2:

"Shear in frame beams and columns and their connections shall be determined on the basis of dead and live loads acting in conjunction with moments equal to the member flexural capacities at critical sections. However, these shears shall not be less than the shears resulting from elastic distribution of the specified forces.

"Beam-column joint panel zone areas shall be designed to resist the computed shear stress, 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."

Applicable definitions of terms remain unchanged.

10A.7.6 Add the following to Sec. J1:

"Each girder moment connection to a column shall be capable of developing in the girder the full bending strength of the girder unless it can be shown that adequate rotation can be obtained by deformation of the connected materials. The connection consists of only those elements that connect the member to the joint. A girder-to-column connection may be considered to develop the bending strength of the girder if the girder flanges have a complete butt weld connection to the column and the girder web is connected to the column through a welded connection or a high strength bolted connection, provided the high-strength bolts are installed to the minimum tension for a slip-critical connection."

10A.7.7 Change the start of the last paragraph in Sec. F1.1 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

11.1 REFERENCE DOCUMENT

The quality and testing of concrete and steel materials and the design and construction of reinforced concrete components that resist seismic forces shall conform to the requirements of the reference 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 318-83, including Appendix A

11.1.1 Modifications to Ref. 11.1

11.1.1.1 Replace Sec. 9.2.3 with Sec. 3.7.1 of this document.

11.1.1.2 Replace Sec. A.2.1.3 and A.2.1.4 with the provisions of this chapter.

11.1.1.3 Amend Sec. A.2.1.5 to read as follows:

> "A reinforced concrete structural system not satisfying the requirements of this appendix, including those composed of precast elements, may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this appendix."

11.1.1.4 Modify Sec. A.2.5.1 to read as follows:

"Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary members shall comply with ASTM A706 except as modified herein. ASTM A615 Grades 40 and 60 ... not less than 1.25. Post-tensioning tendons may be used in flexural members of frames provided the average prestress f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension, does not exceed 350 psi."

Add the following to the end of Sec. A.2.5.1:

"When reinforcing steel is to be welded, the steel shall comply with ASTM A706. This requirement may be satisfied by the use of steel complying with ASTM A615 provided that this steel meets the carbon equivalent requirements and chemical limits for ASTM A706 steel."

➤

11.1.1.5 Insert the following new Sec. A.3.2.3 and change the existing Sec. A.3.2.3 and A.3.2.4 to A. 3.2.4 and A.3.2.5, respectively:

"For members in which prestressing tendons are used together with ASTM A706 or with A615 (Grades 40 or 60) reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more than one quarter of the strength for both positive moments and negative moments at the joint face. Anchorages for tendons must be demonstrated to perform satisfactorily for seismic loadings. Test assemblies shall withstand, without failure, a minimum of 50 cycles of loading ranging between 40 and 85 percent of the minimum specified strength of the tendon. Tendons shall extend through exterior joints and be anchored at the exterior face of the joint or beyond."

11.1.1.6 Change Sec. A.3.3.4 to read as follows:

"Where hoops are not required, stirrups with 135-degree or greater hooks with 6-bar-diameter extensions shall be spaced not more than d/2 throughout the length of the member."

11.1.1.7 Add the following new paragraph to Sec. A.4.4:

"At any section where the nominal strength, ϕP_n, of the column is less than the sum of the shear V_e computed in accordance with Sec. A.7 for all the beams framing into the column above the level under consideration, special transverse reinforcement shall be provided. For beams framing into opposite sides of the column, the moment components may be assumed to be of opposite sign. For determination of the nominal strength, P_n, of the column, these moments may be assumed to result from the deformation of the frame in any one principal axis."

11.1.1.8 In Eq. A-4, Sec., A.4.4.1(2), change the coefficient 0.12 to 0.09.

11.1.1.9 Add the following to the end of Sec. A.4.4.5:

"The special transverse reinforcement shall be placed above the discontinuity for at least the development length of the largest longitudinal reinforcement in the column in accordance with Sec. A.6.4. If the lower end of the column terminates on a wall, the special transverse reinforcement shall extend into the wall for at least the development length of the largest longitudinal reinforcement in the column at the point of termination. If the column terminates on a footing or mat, extend the transverse reinforcement into the footing or mat either the compressive development length of the largest longitudinal reinforcement or the lead length of a standard hook."

11.1.1.10 Add the following to the end of Sec. A.5.1:

"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. Where untopped precast elements are used for diaphragms, the ϕ factor for connections between elements shall be 0.5 except that for connection elements that form a continuous tie across and through the untopped element, extending across the diaphragm, the ϕ factor shall to 0.7."

11.1.1.11 Add the following to the end of Sec. A.5.3.5:

> "Transverse reinforcement terminating at the edges of shear walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U stirrups having the same size and spacing as, and spliced to, the transverse reinforcement. When the nominal shear stress level is below $A_{cv}\sqrt{f'_c}$, this requirement is exempted."

11.1.1.12 Add the following to the end of Sec. A.6.2.2:

> "At these locations, the spacing specified in A.4.4.2(b) may be increased to 6 inches."

11.1.1.13 Change the definition of A_j in Sec. A.6.3.1 (and Sec. A.0) to read:

> "A_j = effective cross-sectional area within a joint, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of: (a) beam width plus the joint depth, (b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side."

11.1.1.14 Change the reference to Sec. 9.2 in Sec. A.7.1.3 to the load combination specified in Sec. 3.7.1 of this document for earthquake sources.

11.1.1.15 Revise Sec. A.8.1 to read:

> "A.8.1 All frame members assumed not to be part of the lateral force resisting system shall satisfy the minimum reinforcement requirements specified in Sec. A.3.2.1 and A.5.2.1 as well as those specified in Chapters 7, 10, and 11."

11.1.1.16 Change the title of Sec. A.9 to read: "Requirements for Intermediate Ductility Frames."

11.1.1.17 For the purpose of these provisions, the term "structural walls" in Ref. 11.1 means "shear walls."

11.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and components shall be determined using the strengths permitted and required in Ref. 11.1 as modified by this chapter.

The maximum loads on anchor bolts shall not exceed those given in Table 11-1. Maximum loads greater than those shown may be used for concrete strengths greater than 3,000 psi when accompanied by substantiating evidence.

TABLE 11-1
Maximum Shear and Tension on Bolts[a]

Diameter (in.)	Minimum Embedment (in.)[b]	Shear (lb)	Tension (lb)
1/4	2-1/2	500	360
3/8	3	1,100	900
1/2	4	1,900	1,700
5/8	5	3,000	2,700
3/4	5-1/2	4,300	4,050
7/8	6	5,900	5,750
1	7	7,700	7,500

[a]Values shown are for minimum concrete compressive strength of 3,000 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.

[b]A minimum embedment of 9 bolt diameters shall be provided for anchor bolts located in the top of columns for buildings located in areas where A_v exceeds 0.15.

Anchor bolts at the tops of columns and similar locations shall be enclosed with not less than two No. 4 or three No. 3 ties located within 4 inches from the top of the column.

11.3 ORDINARY MOMENT FRAMES

Ordinary Moment Frames are frames conforming to the requirements of Ref. 11.1 exclusive of Appendix A.

11.4 INTERMEDIATE MOMENT FRAMES

Intermediate Moment Frames are frames conforming to the requirements of Sec. A.9 of Ref. 11.1 in addition to those requirements for Ordinary Moment Frames.

11.5 SPECIAL MOMENT FRAMES

Special Moment Frames are frames conforming to the requirements of Sec. A.2 through A.7 of Ref. 11.1 in addition to those requirements for Ordinary Moment Frames.

11.6 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any construction permitted in Ref. 11.1 and these provisions.

11.7 SEISMIC PERFORMANCE CATEGORY B

Buildings assigned to Category B shall conform to all the requirements for Category A and to the additional requirements for Category B in other chapters of these provisions.

11.8 SEISMIC PERFORMANCE CATEGORY C

Buildings assigned to Category C shall conform to all the requirements for Category B and to the additional requirements for Category C in other chapters of these provisions as well as to the requirements of this section.

11.8.1 Moment Frames

All moment frames that are part of the seismic resisting system shall be Intermediate Moment Frames, conforming to Sec. 11.4, or Special Moment Frames, conforming to Sec. 11.5

11.8.2 Discontinuous Members

Columns supporting reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement at the spacing s_o as defined in Sec. A.9.5.1 of Ref. 11.1 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as defined in Sec. 11.1.1.9 of this chapter.

11.9 SEISMIC PERFORMANCE CATEGORIES D AND E

Buildings assigned to Category D or E shall conform to all of the requirements for Category C and to the additional requirements of this section.

11.9.1 Moment Frames

All moment frames that are part of the seismic resisting system, regardless of height, shall be Special Moment Frames in conformance with Sec. 11.5.

11.9.2 Seismic Resisting System

All materials and components in the Seismic Resisting System shall conform to Sec. A.2 through A.7 of Ref. 11.1.

11.9.3 Frame Components Not Part of the Seismic Resisting System

All frame components assumed not to be part of the Seismic Resisting System shall conform to Sec. 3.3.4.3 of these provisions and to Sec. A.8 of Ref. 11.1.

Chapter 12

MASONRY

12.1 REFERENCE DOCUMENTS

The design, construction, and quality assurance of masonry components that resist seismic forces shall conform to the requirements of the reference listed in this section except as modified by the provisions of this chapter.

Ref. 12.1 Building Code Requirements for Masonry Structures, ACI-ASCE 530, including Appendix A, "Special Provisions for Seismic Design," and Specifications for Masonry Structures, ACI-ASCE 530.1-88

12.1.1 Modifications to Appendix A of Reference 12.1

Replace all references to seismic zones (ANSI A58.1 zones) with the Seismic Performance Categories listed in Table 12-1.

TABLE 12-1
Appendix A Seismic Zones and Replacement Seismic Performance Categories

Appendix A (ANSI A58.1) Seismic Zone	Replace with Seismic Performance Category
0 and 1	A and B
2	C
3 and 4	D and E

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 except as modified in Sec. 12.2.1 shall be determined using a capacity reduction factor, ϕ, and 2.5 times the allowable working stress determined from Ref. 12.1 including the modifications to the allowable working stress stated therein.

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

12.3 RESPONSE MODIFICATION COEFFICIENTS

The R factors of Table 3-2 for reinforced masonry shall apply, provided masonry is designed in accordance with Ref. 12.1, Chapter 7 and Appendix A. The R factors of Table 3-2 for unreinforced masonry shall apply for all other masonry.

12.4 SEISMIC PERFORMANCE CATEGORY A

Buildings assigned to Category A may be of any type of masonry construction permitted in the reference document.

12.5 SEISMIC PERFORMANCE CATEGORY B

Buildings assigned to Category B shall conform to all the requirements for Category A and the lateral load resisting system shall be designed in accordance with Ref. 12.1, Chapter 6 or 7.

12.6 SEISMIC PERFORMANCE CATEGORY C

Buildings assigned to Category C shall conform to the requirements of Category A; to the requirements Ref. 12.1, Appendix A; and to the additional requirements of this section.

12.6.1 Construction Requirements

12.6.1.1 Multiple Wythe Walls Not Acting Compositely

At least one wythe of a cavity wall shall be designed and reinforced in accordance with Ref. 12.1; the other wythe shall be tied to its backup and reinforced with a minimum of one No. 9 wire gage at a maximum spacing of 16 inches o.c. Wythe shall be tied in accordance with Ref. 12.1, Sec. 5.8.2.2.

12.6.1.2 Screen Walls

Masonry screen walls, laterally supported but not otherwise connected on all edges by a structural frame of concrete masonry or steel, shall meet the following requirements:

12.6.1.2.1 All screen walls shall be reinforced in accordance with this section. 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 joint reinforcement having a minimum steel area of 0.03 square inch. Joint reinforcement shall be embedded in mortar or grout.

12.6.1.2.2 In calculating the resisting capacity of the system, compression and tension in the spaced wires may be utilized. Joint reinforcement shall not be spliced and shall be the widest that the mortar joint will accommodate allowing 1/2 inch of mortar cover.

12.6.2 Material Requirements

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

Structural Clay Loadbearing Wall Tile (ASTM C 34)
Structural Clay Non-Loadbearing Wall Tile (ASTM C 56)

12.7 SEISMIC PERFORMANCE CATEGORY D

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

12.7.1 Construction Requirements for Masonry Laid in Other than Running Bond

The maximum spacing of horizontal reinforcement shall not exceed 24 inches.

12.7.2 Shear Wall Requirements

Shear walls shall comply with the requirements of this section.

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

12.7.2.2 When reinforcement is required in accordance with Ref. 12.1, Sec. 7.5.2, the computed reinforcement shall be placed horizontally.

12.8 SEISMIC PERFORMANCE CATEGORY E

Buildings assigned to Category E shall conform to the requirements of Category D and to the additional requirements and limitations of this section.

12.8.1 Construction Requirements

Construction procedures or admixtures shall be used to minimize cracking of grout and to maximize bond. The thickness of the grout between masonry units and reinforcing shall be a minimum of 1/2 inch for structural masonry.

12.8.1.1 Reinforced Hollow Unit Masonry

Structural reinforced hollow unit masonry shall conform to the following requirement: Vertical reinforcement shall be securely held in position at tops, bottoms, splices, and at intervals not exceeding 112 bar diameters. Horizontal wall reinforcement shall be securely tied to the vertical reinforcement or held in place during grouting by equivalent means.

12.8.1.2 Stacked Bond Construction

All stacked bond construction shall conform to the following requirements:

12.8.1.2.1 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 or 16 inches for structural masonry.

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

12.8.1.2.3 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.

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George W. Gibson, National Elevator Industry, Inc., Farmington, Connecticut

Sheet Metal and Air Conditioning Contractors' National Association

Walter Drown, Sheet Metal and Air Conditioning Contractors' National Association, Los Angeles, California

Structural Engineers Association of California

Eugene Cole, Cole, Yee, Schubert and Associates, Sacramento, California

TC 9, Regulatory Use

Associated General Contractors of America

Cecil Mark, Mark Diversified, Sacramento, California

American Institute of Architects

David Bullen, American Institute of Architects, Washington, D.C.

American Society of Civil Engineers

Norton Remmer (Chairman), Consulting Engineer, Worcester, Massachusetts

Association of Major City Building Officials

Franklin Lew, City of San Francisco, California

Building Officials and Code Administrators International

Bob McCluer, Bldg Officials and Code Administrators International, Country Club Hills, Illinois

Building Seismic Safety Council

Vincent R. Bush (Representative to the TMC), Consultant, Walnut, California

Interagency Committee on Seismic Safety in Construction

G. Robert Fuller, Department of Housing and Urban Development, Washington, D.C.

Appendix

International Conference of Building Officials

Phillip C. Phillips, City of San Leandro, California

National Council of States on Building Codes and Standards

David Conover, National Council of States on Building Codes and Standards, Herndon, Virginia

Southern Building Code Congress International

John R. Battles, Southern Building Code Congress International, Birmingham, Alabama

BUILDING SEISMIC SAFETY COUNCIL MEMBERS

AFL-CIO Building and Construction Trades Department
American Concrete Institute
American Consulting Engineers Council
American Council of Independent Laboratories, Inc.
American Institute of Architects
American Institute of Steel Construction
American Insurance Services Group, Inc.
American Iron and Steel Institute
American Plywood Association
American Society of Civil Engineers
Applied Technology Council
Associated General Contractors of America
Association of Engineering Geologists
Association of Major City Building Officials
Association of the Wall and Ceiling Industries, International
Brick Institute of America
Building Officials and Code Administrators, International
Building Owners and Managers Association, International
California Geotechnial Engineers Association
Canadian National Committee on Earthquake Engineering
Concrete Masonry Association of California and Nevada
Concrete Reinforcing Steel Institute
Earthquake Engineering Research Institute
General Reinsurance Corporation *
Interagency Committee on Seismic Safety in Construction
International Conference of Building Officials
Masonry Institute of America
Masonry Institute of Washington
Metal Building Manufacturers Association
National Association of Home Builders
National Association of Housing and Redevelopment Officials
National Concrete Masonry Association
National Conference of States on Building Codes and Standards
National Elevator Industry, Inc.
National Fire Sprinkler Association
National Forest Products Association
National Institute of Building Sciences
National Ready Mixed Concrete Association
Oklahoma Masonry Institute
Permanent Commission for Structural Safety of Buildings *
Portland Cement Association
Post-Tensioning Institute
Prestressed Concrete Institute
Rack Manufacturers Institute
Southern Building Code Congress International
Steel Deck Institute, Inc.
Steel Plate Fabricators Association, Inc.
Steven Winter Associates, Inc. *
Structural Engineers Association of Arizona
Structural Engineers Association of California
Structural Engineers Association of Central California
Structural Engineers Association of Illinois
Structural Engineers Association of Northern California
Structural Engineers Association of San Diego
Structural Engineers Association of Southern California
Structural Engineers Association of Utah
Structural Engineers Association of Washington
The Masonry Society
Western States Council Structural Engineers Association
Western States Clay Products Association

* Affiliate Members