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1991 Edition

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

Part 1
Provisions

EARTHQUAKE HAZARDS REDUCTION SERIES 16

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BUILDING
SEISMIC
SAFETY
COUNCIL

A council of the National Institute of Building Sciences

Program
on
Improved
Seismic
Safety
Provisions

1991 Edition

NEHRP RECOMMENDED PROVISIONS FOR THE DEVELOPMENT OF SEISMIC REGULATIONS FOR NEW BUILDINGS

Part 1

Provisions

THE BUILDING SEISMIC SAFETY COUNCIL AND ITS PURPOSE

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of the National Institute of Building Sciences (NIBS) as an entirely new type of instrument for dealing with the complex regulatory, technical, social, and economic issues involved in developing and promulgating building earthquake hazard mitigation regulatory provisions that are national in scope. By bringing together in the BSSC all of the needed expertise and all relevant public and private interests, it was believed that issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC is an independent, voluntary membership body representing 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, elected 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 types, structures, and related facilities and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. The BSSC 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 (i.e., 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 -promulgating role; rather, it advocates that code- and standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

BSSC Program on Improved Seismic Safety Provisions

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

1991 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.
1991

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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 documents listed below; unless otherwise noted, single copies are available at no charge from the Council:

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, 1990

NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1991 Edition, 2 volumes and maps, 1991

Non-Technical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1990

Seismic Considerations for Communities at Risk, 1990*

Seismic Considerations: Elementary and Secondary Schools, Revised Edition 1990

Seismic Considerations: Health Care Facilities, Revised Edition, 1990

Seismic Considerations: Hotels and Motels, Revised Edition, 1990

Seismic Considerations: Apartment Buildings, 1988

Seismic Considerations: Office Buildings, 1988

Societal Implications: Selected Readings, 1986.

Strategies and Approaches for Implementing a Comprehensive Program to Mitigate the Risk to Lifelines from Earthquakes and Other Natural Hazards, 1989 (available from the National Institute of Building Sciences for $11)

For further information concerning any of these documents or the activities of the BSSC, contact the Executive Director, Building Seismic Safety Council, 1201 L St., N.W., Suite 400, Washington, D.C. 20005.

*Earlier versions of this publication were entitled *Societal Implications: A Community Handbook*.

PREFACE

Publication of the 1991 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 second in a planned updating of both the *Provisions* and a set of complementary publications. As in the case of the first edition of the Provisions in 1985, FEMA encourages widespread dissemination and voluntary utilization of the technology contained in the 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 1991 Edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* is the third edition of the document and, like the 1985 and 1988 Editions that preceded it, has the consensus approval of the Building Seismic Safety Council membership. 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, the Appendix of the *Commentary* volume presents a detailed description of the effort.)

During the effort to update the 1988 Edition for issuance as the 1991 Edition, the BSSC Provisions Update Committee (PUC), its 11 technical subcommittees, and the BSSC Board of Direction (see Appendix B of this volume for a list of members of these groups) examined issues left unresolved when the 1988 Edition was published and considered new experience and research data that had become available. Their work resulted in proposals for change to the 1988 Edition that were balloted by the Council membership in April 1991. The results of this balloting highlighted several somewhat controversial issues and prompted the PUC and Board to submit a number of revised proposals to the membership for reballot in July 1991. This 1991 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 the two earlier 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 1988 and 1991 Editions of the *Provisions* is presented in Appendix A of this volume.

In presenting this 1991 Edition of the *NEHRP Recommended Provisions*, the BSSC wishes to acknowledge the accomplishments of the many individuals and organizations involved over the years. The BSSC program resulting in the first two editions of the *Provisions*, the 1991 update effort, and the information development/dissemination activities conducted to stimulate use of the *Provisions* has benefitted from the expertise of hundreds of specialists, many of whom have given freely of their time over many years.

With so many volunteers participating, it is difficult to single out a given number or group for special recognition without inadvertently omitting others without whose assistance the BSSC program could not have succeeded; nevertheless, the 1991 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 Provisions Update Committee, especially Chairman Loring Wyllie;

- The members of the 11 PUC Technical Subcommittees; 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 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 to all those who provided advice, counsel, and encouragement during conduct of the 1991 update effort or who otherwise participated in the BSSC program that resulted in the *NEHRP Recommended Provisions*.

Gerald Jones
Chairman, BSSC Board of Direction

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NOTE

Those portions of the 1991 Edition of the *Provisions* that are substantively different from the 1988 Edition are identified in the margins as follows:

additions
or
revisions

deletions

Not highlighted are editorial and format changes.

A summary of the differences between the 1988 and 1991 Editions of the *Provisions* is presented in Appendix A of this 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: Every building, and portion thereof, shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Additions to existing buildings also shall be designed and constructed to resist the effects of earthquake motions determined as prescribed by these provisions. Existing buildings and alterations and repairs to existing buildings need only comply with these provisions when required by Sec. 1.3.1 through 1.3.3.

EXCEPTIONS:

1. Detached one- and two-family dwellings that are located in seismic map areas having an effective peak velocity-related acceleration (A_v) value less than 0.15 are exempt from the requirements of these provisions.

2. Agricultural storage buildings that are intended only for incidental human occupancy are exempt from the requirements of these provisions.

3. Buildings located in seismic map areas having an effective peak velocity-related acceleration (A_v) value less than 0.05 shall only be required to comply with Sec. 3.6.1.

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

1.3 APPLICATION OF PROVISIONS: 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 building 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 quality assurance requirements of Sec. 1.6. The analysis and design of structural systems and components, including foundations, frames, walls, floors and roofs, shall be in accordance 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 including tenant improvements shall be designed in accordance with Chapter 8.

> **EXCEPTION:** Detached one- and two-family wood frame dwellings with a building height of not more than 2 stories or 35 feet that are located in seismic map areas having an effective peak velocity-related acceleration (A_v) value equal to or greater than 0.15 are only required to be constructed in accordance with Sec. 9.8.

1.3.2 ADDITIONS TO EXISTING BUILDINGS: Additions shall be made to existing buildings only as follows:

1.3.2.1: An addition that is structurally independent from an existing building shall be designed and constructed in accordance with the seismic requirements herein.

1.3.2.2:* An addition that is not structurally independent from an existing building shall be designed and constructed such that the entire building conforms to the seismic force resistance requirements for new buildings unless the following three conditions are complied with:

1. The addition shall comply with the requirements for new buildings, and

2. The addition shall not increase the seismic forces in any structural element of the existing building by more than 5 percent unless the capacity of the element subject to the increased forces is still in compliance with these provisions, and

3. The addition shall not decrease the seismic resistance of any structural element of the existing building unless the reduced seismic resistance of the element is equal to or greater than that required for new buildings.

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*The changes to this section were approved by the required two-thirds majority of BSSC members during balloting; however, several members raised issues that remain in need of attention during the next updating of the *Provisions*. These members maintain that Item 3 should address the entire structure rather than individual elements.

1.3.3 CHANGE OF USE: When a change of use results in a building being reclassified to a higher Seismic Hazard Exposure Group, the building shall conform to the seismic requirements for new construction.

> **EXCEPTION:** When a change of use results in a building being reclassified from Seismic Hazard Exposure Group I to Seismic Hazard Exposure Group II, compliance with these provisions is not required if the building is located in a seismic map area having an effective peak velocity-related acceleration (A_v) value of less than 0.15.

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 SEISMIC GROUND ACCELERATION MAPS:* The effective peak acceleration (A_a) and the effective peak velocity-related acceleration (A_v) shall be determined from Maps 1 and 2, respectively. Where site-specific ground motions are used or required, they shall be developed with 90 percent probability of the ground motion not being exceeded in 50 years.

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

TABLE 1.4.1.1
Coefficients A_a and A_v

Map Area from Map 1 (A_a) or Map 2 (A_v)	Value of A_a and A_v
7	0.40
6	0.30
5	0.20
4	0.15
3	0.10
2	0.05
1	< 0.05 [a]

[a] For equations or expressions incorporating the terms A_a or A_v, a value of 0.05 shall be used.

*New maps and a new approach (method) for establishing design ground motions are presented in the "Appendix to Chapter 1" and the *Commentary* section on that appendix. Since the provisions in this appendix are preliminary, they *should not* be used for design at this time; rather, it is hoped that they will be evaluated in trial designs and independently reviewed by appropriate structural engineering and seismological technical committees.

1.4.1.2: Alternatively, values of A_a and A_v may be determined directly from Maps 3 and 4, respectively; interpolation may be used in reading Maps 3 and 4 or the higher adjacent value shall be used.

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 (A_a) and effective peak velocity-related acceleration (A_v) The coefficients A_a and A_v to be used in the application of these provisions are established as:

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:

1.4.2.1 Group III: Seismic Hazard Exposure Group III buildings are those having essential facilities that are required for post-earthquake recovery including:

1. Fire or rescue and police stations
2. Hospitals or other medical facilities having surgery or emergency treatment facilities
3. Emergency preparedness centers including the equipment therein
4. Power generating stations or other utilities required as emergency back-up facilities for Seismic Hazard Exposure Group III facilities
5. Emergency vehicle garages
6. Communication centers
7. Buildings containing sufficient quantities of toxic or explosive substances deemed to be dangerous to the public if released

1.4.2.2 Group II: Seismic Hazard Exposure Group II buildings are those that have a substantial public hazard due to occupancy or use including:

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

4. Medical facilities with 50 or more resident incapacitated patients but not having surgery or emergency treatment facilities

5. Jails and detention facilities

6. All structures with an occupancy greater than 5,000 persons

7. Power generating stations and other public utility facilities not included in Seismic Hazard Exposure Group III and required for continued operation

1.4.2.3 Group I: Seismic Hazard Exposure Group I buildings are those not assigned to Seismic Hazard Exposure Group III or Group II.

1.4.2.4 Multiple Use: Buildings having multiple uses shall be assigned the classification of the highest Seismic Hazard Exposure Group present.

1.4.2.5 Group III Building Protected Access: Where operational access to a Seismic Hazard Exposure Group III building is required through an adjacent building, the adjacent building shall conform to the requirements for Group III buildings. Where operational access is less than 10 feet from the interior lot line or another building on the same lot, protection from potential falling debris from adjacent buildings shall be provided by the owner of the Seismic Hazard Exposure Group III building.

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. Site-specific conditions as specified in Sec. 8.3.7 that could result in the interruption of utility services shall be considered when providing the capacity to continue to function.

1.4.3 SEISMIC PERFORMANCE CATEGORY: Buildings shall be assigned a Seismic Performance Category in accordance with Table 1.4.3.

TABLE 1.4.3
Seismic Performance Category

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

1.4.4 SITE LIMITATION FOR SEISMIC PERFORMANCE CATEGORY E: A building assigned to Category E shall not 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 shall be submitted demonstrating that the proposed alternate, for the purpose intended, will be at least equal in strength, durability, and seismic resistance.

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. The quality assurance provisions apply to the following:

1. The seismic force resisting system in buildings assigned to Category C, D, or E.

2. Other designated seismic systems in buildings assigned to Category E.

3. All other buildings when required by the regulatory agency.

1.6.1 QUALITY ASSURANCE PLAN: A quality assurance plan shall be submitted to the regulatory agency.

1.6.1.1 Details of Quality Assurance Plan: The quality assurance plan shall specify the designated seismic systems or seismic force resisting system in accordance with Sec. 1.6 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 contain 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 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 quality assurance plan for the following construction work:

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:* Periodic special inspection during and upon completion of the placement of steel in reinforced concrete special moment frames.

1.6.2.2.2: Periodic special inspection of 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: Periodic special inspection is required during the placement of prestressing steel and continuous special inspection is required during all 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:

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.

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*The changes to this section were approved by the required two thirds majority of BSSC members during balloting; however, several members raised issues that remain in need of attention during the next updating of the *Provisions*. Those concerned believe that in Sec. 1.6.2.2.1 for reinforced concrete special moment frames, inspection should be essentially continuous during the placing of reinforcing steel while work is in progress.

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 approved plans at the completion of welding.

1.6.2.6.2: Periodic special inspection is required in accordance with Ref. 10.1 or 10.2 for installation and tightening of fully tensioned high-strength bolts in slip-critical connections and in connections subject to direct tension.

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 having a performance criteria factor (P) equal to 1 or 1.5 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 having a performance criteria factor (P) equal to 1 or 1.5:

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 weldability, elongation, yield strength, and ultimate strength.

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 318-89.

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 D1.1-90. 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 as established by the person responsible for the structural design.

1.6.3.5 Mechanical and Electrical Equipment: Each manufacturer of components utilized in a designated seismic system with a performance criteria factor (*P*) stated for the component in Chapter 8 as equal to 1 or 1.5 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. 8.3.5 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 designated seismic system in a building of Category E where the performance criteria factor (*P*) stated for the equipment in Chapter 8 is equal to 1 or 1.5 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

PRELIMINARY SPECTRAL RESPONSE MAPS AND METHOD FOR ESTABLISHING DESIGN GROUND MOTIONS

PREFACE: The NEHRP Recommended Provisions is a resource document, not a model code. This appendix is presented to introduce to potential users a new and relevant technique and maps for estimated spectral response accelerations. The provisions are preliminary, but they are included in order to obtain broad professional review and comment. The maps and technique were developed as a result of a collaborative effort between seismologists and practicing structural engineers who have been working on improving descriptions of earthquake motions for use by designers. Maps for 90 percent probability of nonexceedance in 250 years are presented for qualititative information only because of the large uncertainties involved. Review and comment on the appendix and maps are invited.

Maps 5 through 12 are provided for use with this appendix. They provide a different measure of the seismic ground shaking hazard than Maps 1 through 4. This appendix describes how Maps 5 through 8 are to be used with the remainder of these provisions. Maps 9 through 12 are provided to show how the ground-motion parameters vary when the exposure time is increased significantly over that used for Maps 1 through 8. Maps 9 through 12 are intended to be advisory only.

Use of Maps 5 through 12 to determine Seismic Performance Categories or index values (such as the use of A_v in Sec. 1.2) requires further study. Accordingly, no such conversions are given in this appendix. The appendix does contain all conversions necessary to compute seismic design forces. In order to use the new maps, follow the steps outlined below.

1. Replace the definitions for the acceleration coefficients and symbols A_a and A_v with the following:

   Spectral Response Acceleration:

   Spectral Response Acceleration (0.3): The spectral response acceleration coefficient, $S_{A(0.3)}$, at a period of 0.3 second expressed as a fraction of gravity for use in determining the prescribed seismic forces given in Sec. 1.4.

   Spectral Response Acceleration (1.0): The spectral response acceleration coefficient, $S_{A(1.0)}$, at a period of 1.0 second expressed as a fraction of gravity for use in determining the prescribed seismic forces given in Sec. 1.4.

2. Revise Sec. 1.4.1 as follows:

1.4.1 SEISMIC GROUND ACCELERATION MAPS: The spectral response acceleration (0.3), $S_{A(0.3)}$, and the spectral response acceleration (1.0), $S_{A(1.0)}$, shall be determined from Maps 5, 6, 7, or 8, respectively. Interpolation shall be permitted in the determination of the spectral accelerations. These maps were developed with 90 percent probability of the ground motions not being exceeded in 50 years. Where site-specific ground motions are used or required, they shall be developed on the same basis. Maps 9, 10, 11, and 12 show how the spectral response accelerations vary when the exposure period is increased significantly over that used for Maps 5, 6, 7, and 8.

3. Delete Sec. 1.4.1.1 and Table 1.4.1.1.

4. Delete Sec. 1.4.1.2.

5. Revise Alternate Sec. 1.4.1 as follows:

ALTERNATE SEC. 1.4.1 FOR REGULATORY AGENCIES THAT HAVE MADE A DETERMINATION OF $S_{A(03)}$ AND $S_{A(1.0)}$: The design ground motions are defined in terms of spectral response acceleration (0.3), $S_{A(0.3)}$, and spectral response acceleration (1.0), $S_{A(1.0)}$. The coefficients $S_{A(0.3)}$ and $S_{A(1.0)}$ to be used in the application of these provisions are established as:

$S_{A(0.3)}$ = ___ and $S_{A(1.0)}$ = ___.

6. Revise Table 3.2 as follows:

TABLE 3.2
Site Coefficient

Soil Profile Type	Description	Site Coefficient, S
S_1	A soil profile with either: (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.	0.8
S_2	A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.0
S_3	A soil profile containing 20 to 40 feet in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils.	1.3
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts.	1.7

7. Revise the first paragraph of Sec. 3.6.1.1 as follows:

   3.6.1.1 Connections: 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. Any smaller portion of the building shall be tied to the remainder of the building with elements having a strength of 1/2 of the spectral acceleration (1.0), $S_{A(1.0)}$, times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater.

8. Revise Sec. 3.6.1.2 as follows:

   3.6.1.2 Anchorage of Concrete or Masonry Walls: 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 the greater of a seismic lateral force (F_p) induced by the wall or 700 times the spectral acceleration (1.0), $S_{A(1.0)}$, (pounds) per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

9. Revise the second paragraph of Sec. 3.6.2.7 as follows:

   Floor and roof diaphragms shall be designed to resist the following seismic forces: A minimum force equal to 35 percent of the spectral acceleration (1.0), $S_{A(1.0)}$, 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 force resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

10. Revise Sec. 3.6.2.8 as follows:

    3.6.2.8 Bearing Walls: Exterior and interior bearing walls and their anchorage shall be designed for a force equal to 70 percent of the spectral acceleration (1.0), $S_{A(1.0)}$, times the weight of wall (W_c), normal to the surface, with a minimum force of 10 percent of the weight of the wall. Interconnection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

11. In Sec. 3.7, replace the term $0.5A_v$ in Eq. 3-1, 3-1a, 3-2, and 3-2a with the term $0.3S_{A(1.0)}$.

12. Revise the last sentence of the last paragraph of Sec. 3.7 as follows:

    In Eq. 3-1, 3-1a, 3-2, and 3-2a, the term $0.4S_{A(1.0)}$ may be neglected where $S_{A(1.0)}$ is equal to or less than 0.03.

13. In Sec. 4.2.1, revise Eq. 4-2 and its definitions as follows:

$$C_s = \frac{S_{A(1.0)}S}{RT^n} \tag{4-2}$$

where

$S_{A(1.0)}$ = the coefficient representing spectral acceleration (1.0) from Sec. 1.4.1,

S = the coefficient for the soil profile characteristics of the site in Table 3.2,

R = the response modification factor in Table 3.3, and

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

n = 1.0 for $T \leq 1.0$ sec and 2/3 for $T > 1.0$ sec.

14. In Sec. 4.2.1, revise Eq. 4-3 and its definitions as follows:

$$C_s = \frac{S_{A(0.3)}}{R} \tag{4-3}$$

where:

$S_{A(0.3)}$ = the seismic coefficient representing the spectral acceleration (0.3) as determined in Sec. 1.4.1.

R = the response modification factor in Table 3.3.

15. Revise Table 4.2.2 as follows:

TABLE 4.2.2
Coefficient for Upper Limit on Calculated Period

Coefficient Representing Spectral Acceleration (1.0), $S_{A(1.0)}$	Calculated Period (C_a)
0.60	1.2
0.45	1.3
0.30	1.4
0.20	1.5
0.15	1.7
0.075	1.7

16. In Sec. 5.5, revise Eq. 5-3 and its definitions as follows:

$$C_{sm} = \frac{S_{A(1.0)} S}{R T_m^n} \quad (5\text{-}3)$$

where:

$S_{A(1.0)}$ = the coefficient representing spectral acceleration (1.0) from Sec. 1.4.1,

S = the coefficient for the soil profile characteristics of the site as determined from Table 3.1,

R = the response modification factor determined from Table 3.3,

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the building, and

n = 1.0 for $T_m \leq 1.0$ sec and 2/3 for $T_m > 1.0$ sec.

17. In Sec. 5.5, revise Exceptions 2 and 3 as follows:

2. For buildings on sites soil profile characteristics *S3* or *S4*, the modal seismic design coefficient, *Csm*, for modes other than the fundamental mode that have periods less than 0.3 seconds is permitted to be determined by the following equation:

$$C_{sm} = \frac{(S_{A(0.3)}/2.5)}{R}(1.0 + 5.0T_m) \quad (5\text{-}3a)$$

3. For buildings where any modal period of vibration (T_m) exceeds 4.0 seconds, the modal seismic design coefficient (C_{sm}) for that mode is permitted to be determined by the following equation:

$$C_{sm} = \frac{2 S_{A(1.0)} S}{R T_m^{4/3}} \quad (5\text{-}3b)$$

where:

$S_{A(0.3)}$ = the seismic coefficient representing the spectral acceleration (0.3) as determined in Sec. 1.4.1.

$S_{A(1.0)}$ = the coefficient representing spectral acceleration (1.0) from Sec. 1.4.1,

R = the response modification factor determined from Table 3.3,

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the building, and

S = the coefficient for the soil profile characteristics of the site as determined from Table 3.1.

18. Revise Sec. 7.4.3 as follows:

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 the spectral acceleration (1.0), $S_{A(1.0)}$, divided by 6 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.

19. Revise Eq. 8-1 and its definitions as follows:

$$F_p = \frac{S_{A(1.0)}}{1.5} C_c P W_c \tag{8-1}$$

where:

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

$S_{A(1.0)}$ = the coefficient representing spectral acceleration (1.0) from Sec. 1.4.1,

C_c = the seismic coefficient for architectural components from Table 8.2.2,

P = the performance criteria factor from Table 8.2.2, and

W_c = the weight of the architectural component.

20. Revise Eq. 8-2 and its definitions as follows:

$$F_p = \frac{S_{A(1.0)}}{1.5} C_c P a_c W_c \tag{8-2}$$

where:

$S_{A(1.0)}$ = the coefficient representing spectral acceleration (1.0) from Sec. 1.4.1,

C_c = the seismic coefficient for mechanical and electrical components from Table 8.3.2a,

P = the performance criteria factor from Table 8.3.2a,

a_c = the amplification factor determined in accordance with Table 8.3.2b, and

W_c = The operating weight of the mechanical or electrical component or system.

21. Revise Sec. 8.3.6.1 as follows:

8.3.6.1 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 spectral acceleration (1.0), $S_{A(1.0)}$, equal to or greater than 0.20 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 0.35 times the spectral acceleration (1.0), $S_{A(1.0)}$.

Chapter 2

DEFINITIONS AND SYMBOLS

2.1 DEFINITIONS: The definitions presented in this section provide the meaning of the terms used in these provisions. Definitions of terms that have a specific meaning relative to the use of wood, steel, concrete, or masonry are presented in the chapter devoted to the material (Chapters 9 through 12, respectively).

Acceleration:

Effective Peak Acceleration: A coefficient representing ground motion at a period of about 0.1 to 0.5 second (A_a) as determined from Sec. 1.4.1.

Effective Peak Velocity-Related Acceleration: A coefficient representing ground motion at a period of about 1.0 second (A_v) as determined from Sec. 1.4.1.

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.

Component Supporting Mechanism: The structural means by which mechanical and electrical components and systems transfer seismically induced loads to the building.

Fixed or Direct Connection: A supporting mechanism wherein the principal load transfer is characterized by nominally small displacements such as shear or axial deformation of support structural elements, direct bearing on the building, and shear or tensile deformation of connecting bolts.

Resilient Support System: A supporting mechanism wherein the principal load transfer is provided by elements that are clearly more flexible than the component such as support structural elements loaded in flexure, springs, and rubberized or fibrous mounts.

Seismic Activated Restraining Device: An interactive restraining device that is activated by or provides resistance to earthquake motion.

Confined Region: That portion of a reinforced concrete component in which the concrete is confined by closely spaced special transverse 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.

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 force 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 force resisting system.

Frame:

Braced Frame: 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 (CBF): 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.

Intermediate Moment Frame (IMF): A moment 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. 11.3.2.

Ordinary Moment Frame (OMF): A moment 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, Sec. 11.3.1, or Sec. 12.3.

Special Moment Frame (SMF): A moment 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. 10.10, Sec. 11.3.3, or Sec. 12.3.

Frame System:

Building Frame System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by shear walls or braced frames.

Dual Frame System: A structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by a moment resisting frame and shear walls or braced frames as prescribed in Sec. 3.3.1.

Space Frame System: 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.

High Temperature Energy Source: A fluid, gas, or vapor whose temperature exceeds 220 degrees F.

Inspection, Special: The observation of the work by the special inspector to determine compliance with the approved design documents and these provisions.

Continuous Special Inspection: The full-time observation of the work by an approved special inspector who is present in the area where work is being performed.

Periodic Special Inspection: 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.

Inverted Pendulum Type Structures: Structures that have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. The structures are usually T-shaped with a single column supporting the beams or slab at the top.

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

Load:

Dead Load (Q_D): 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 Load (W): The total dead load and applicable portions of other loads as defined in Sec. 4.2.

Live Load (Q_L): The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load. See Sec. 4.2. The live load may be reduced for tributary area as permitted by the building code administered by the regulatory agency.

Snow Load (Q_S): A vertical load due to the weight of the accumulation of snow. See Sec. 4.2.

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.

Roofing Unit: A unit of roofing material weighing more than 1 pound.

Seismic Force 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.

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.

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

Site Coefficient: A coefficient assigned to a building site based on soil properties as defined in Sec. 3.2.

Special Transverse 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 Wall: An exterior or interior wall providing support for vertical loads.

Cripple Wall: Short stud wall between the foundation and the lowest framed floors with studs not less than 14 inches long.

Light Framed Wall: A wall with wood or steel studs.

Nonbearing Wall: An exterior or interior 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.

Shear Wall: 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 as indicated.

A_a The seismic coefficient representing the effective peak acceleration as determined in Sec. 1.4.1.

A_b	Area (in.^{2}) of bolt or stud in Chapter 11.
A_b	Cross-sectional area of an anchor bolt (in.^{2}) in the "Appendix to Chapter 12."
A_{ch}	Cross-sectional area of a component measured to the outside of the special lateral reinforcement.
A_e	Effective net area (in.^{2}) in Chapter 10.
A_f	Flange area of member (in.^{2}) in Chapter 10.
A_g	Gross cross-sectional area of masonry (in.^{2}) in Chapter 10 and 12.
A_{mv}	Net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered (in.^{2}) in the "Appendix to Chapter 12."
A_n	Net cross-sectional area of masonry (in.^{2}) in the "Appendix to Chapter 12."
A_o	The area of the load-carrying foundation.
A_p	Projected area, on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations (in.^{2}) in the "Appendix to Chapter 12."
A_s	The sloping area (in.^{2}) of an assumed failure surface in Chapter 11.
A_s	Cross-sectional area of reinforcement (in.^{2}) in the "Appendix to Chapter 12."
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 (in.^{2}).
A_{st}	Area of link stiffener (in inches) in Chapter 10.
A_t	The area (in.^{2}) of the flat bottom of the truncated pyramid of an assumed concrete failure surface in Chapter 11.
A_v	The seismic coefficient representing the effective peak velocity-related acceleration as determined in Sec. 1.4.1.
A_{vd}	Required area of leg of diagonal reinforcement.
A_w	Effective area of weld (in.^{2}) in Chapter 10.

A_w Link web area (in.^{2}) in Chapter 10.

A_x The torsional amplification factor.

a_b Length of compressive stress block (in inches) in the "Appendix to Chapter 12."

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.

B_a Nominal axial strength of an anchor bolt (lb) in the "Appendix to Chapter 12."

B_v Nominal shear strength of an anchor bolt (lb) in the "Appendix to Chapter 12."

b Width of compression element (in.) in Chapter 10.

b Width of EBF link stiffener (in.) in Chapter 10.

b Effective width of a member (in.) in the "Appendix to Chapter 12."

b_a Design axial force on an anchor bolt (lb) in the "Appendix to Chapter 12."

b_{cf} Column flange width (in.) in Chapter 10.

b_f Flange width (in.) in Chapter 10.

b_v Design shear force on an anchor bolt (lb) in the "Appendix to Chapter 12."

b_w Web width (in.) in the "Appendix to Chapter 12."

C_a Coefficient for upper limit on calculated period; see Table 4.2.2.

C_c The seismic coefficient for components of buildings as specified in Tables 8.2.2 and 8.3.2a (dimensionless).

C_d The deflection amplification factor as given in Table 3.3.

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 building period coefficient in Sec. 4.2.2.
C_{vx}	The vertical distribution factor as determined in Sec. 4.3.
c	Distance from the neutral axis of a flexural member to the fiber of maximum compressive strain.
D_s	The total depth of the stratum in Eq. 6A-10.
d	Overall depth of member (in.) in Chapters 10 and 12.
d_b	Overall beam depth (in.) in Chapter 10.
d_b	Diameter of reinforcement (in.) in the "Appendix to Chapter 12."
d_c	Overall column depth (in.) in Chapter 10.
d_e	Distance from the anchor axis to the free edge in Chapter 11.
d_z	Overall panel zone depth between continuity plates (in.) in Chapter 10.
E_m	Chord modulus of elasticity of masonry (psi) in the "Appendix to Chapter" 12.
E_s	Modulus of elasticity of reinforcement (psi) in the "Appendix to Chapter 12."
E_v	Shear modulus of masonry (psi) in the "Appendix to Chapter 12."
e	Eccentricity of P_{ou} (in.) in the "Appendix to Chapter 12."
e	EBF link length (in.) in Chapter 10.
F_{BM}	Nominal strength of the base material to be welded (ksi) in Chapter 10
F_{EXX}	Classification strength of weld metal (ksi) in Chapter 10.
F_i, F_n, F_x	The portion of the seismic base shear, V, induced at Level i, n, or x, respectively, as determined in Sec. 4.3.
F_p	The seismic force acting on a component of a building as determined in Sec. 3.7, 8.2, or 8.3.
F_w	Nominal strength in weld (ksi) in Chapter 10.

F_{xm} — The portion of the seismic base shear, V_m, induced at Level x as determined in Sec. 5.5.

F_y — Specified minimum yield stress of the type of steel being used (ksi) in Chapter 10.

F_{yb} — F_y of a beam (ksi) in Chapter 10.

F_{yc} — F_y of a column (ksi) in Chapter 10.

f_c' — Concrete strength, 6,000 psi limit for design.

f_m' — Ultimate compressive strength of masonry in the "Appendix to Chapter 12."

f_r — Modulus of rupture of masonry (psi) in the "Appendix to Chapter 12."

f_s' — Ultimate tensile strength (psi) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, may be assumed to be 60,000.

f_y — Specified yield strength of reinforcement (psi) in the "Appendix to Chapter 12."

f_y — Yield stress of diagonal reinforcement.

f_{yh} — 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.

H — Average story height above and below a beam-to-column connection (in.) in Chapter 10.

h_c — Assumed web depth for stability (in.) in Chapter 10.

$\bar{h}$ — The effective height of the building as determined in Sec. 6A.2 or 6A.3.

h — Height of a wood shear panel or diaphragm in Chapter 9.

h — Effective height of the wall between points of support (in.) in the "Appendix to Chapter 12."

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_{cr}	Moment of inertia of the cracked section (in.^{4}) in the "Appendix to Chapter 12."
I_n	Moment of inertia of the net cross-sectional area of a member (in.^{4}) in the "Appendix to Chapter 12."
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.
k_v	Shear buckling coefficient in Chapter 10.
$\bar{k}$	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	Length of bracing member (in.) in Chapter 10.
L	Length of coupling beam between coupled shear walls in the "Appendix to Chapter 12."
L_o	The overall length of the side of the foundation in the direction being analyzed, Sec. 6A.2.1.
l	Length of a wood shear panel or diaphragm in Chapter 9.
ℓ_b	Effective embedment length of anchor bolt (in.) in the "Appendix to Chapter 12."

ℓ_{be}	Anchor bolt edge distance (in.) in the "Appendix to Chapter 12."
ℓ_d	Development length (in.) in the "Appendix to Chapter 12."
ℓ_{dh}	Equivalent development length for a standard hook (in.) in the "Appendix to Chapter 12."
ℓ_{ld}	Minimum lap splice length (in.) in the "Appendix to Chapter 12."
ℓ_u	Unsupported length of compression member (in.) in the "Appendix to Chapter 12."
ℓ_w	Length of wall (in.) in the "Appendix to Chapter 12."
M	Moment on a masonry section due to unfactored design loads (in.-lb) in the "Appendix to Chapter 12."
M_a	Maximum moment in a member at stage deflection is computed (in.-lb) in the "Appendix to Chapter 12."
M_{cr}	Cracking moment strength of the masonry (in.-lb) in the "Appendix to Chapter 12."
M_d	Design moment strength (in.-lb) in the "Appendix to Chapter 12."
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_n	Nominal moment strength of a member or joint (kip-inches) in Chapter 10.
M_p	Plastic bending moment (kip-inches) in Chapter 10.
M_{pa}	Plastic bending moment modified by axial load ratio (kip-inches) in Chapter 10.
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_u	Required flexural strength on a member or joint (kip-inches) in Chapter 10.
M_u	Required moment strength (in.-lb).

M_1, M_2 Nominal moment strength at the ends of the coupling beam (lb) in the "Appendix to Chapter 12."

N_v Force acting normal to the shear force (lb).

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. 4.2.2.1.

N_v Force acting normal to shear surface (lb) in the "Appendix to Chapter 12."

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

P The performance criteria factor as given in Chapter 8 (dimensionless).

P Axial load on a masonry section due to unfactored loads (lb) in the "Appendix to Chapter 12."

P_D Required axial strength on a column resulting from application of dead load (D) (kips) in Chapter 10.

P_E Required axial strength on a column resulting from application of the amplified earthquake load (E') (kips) in Chapter 10.

P_L Required axial strength on a column resulting from application of live load (L) (kips) in Chapter 10.

P_n Nominal axial load strength (lb) in Chapter 10.

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

P_n Nominal axial strength of a column (kips) in the "Appendix to Chapter 12."

P_u Required axial strength.

$P_u V_u$ Tensile, shear strength required due to factored loads (lb) in Chapter 11.

P_u^* Required axial strength on a brace (kips) in Chapter 10.

P_{uc} Required axial strength on a column based on Eq. 3-1 and 3-2 (kips) in Chapter 10.

P_x	The total unfactored vertical design load at and above Level x.
P_y	Nominal yield axial strength of a member = $F_y A_g$ (kips) in Chapter 10.
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 3.3.
R_n	Nominal strength of a member or joint in Chapter 10.
r	A characteristic length of the foundation as defined in Sec. 6A.2.1.
r	Governing radius of gyration (in.) in Chapter 10.
r	Radius of gyration (in.) in the "Appendix to Chapter 12."
r_a	The characteristic foundation length defined by Eq. 6A-8.
r_m	The characteristic foundation length as defined by Eq. 6A-8.
r_y	Radius of gyration about y axis (in.) in Chapter 10.
S	The coefficient for the soil profile characteristics of the site as given in Table 3.2.
S	Section modulus (in.^{3}) in the "Appendix to Chapter 12."
S_1, S_2, S_3, S_4	The Soil Profile Types as defined in Sec. 3.2.
s	Spacing of shear reinforcement (inches) in the "Appendix to Chapter 12."
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.
t	Thickness of link stiffener (in inches) in Chapter 10.
t	Wall thickness or least lateral dimensions of column (in.) in the "Appendix to Chapter 12."
t_{bf}	Thickness of beam flange (in.) in Chapter 10.
t_{cf}	Thickness of column flange (in.) in Chapter 10.
t_f	Thickness of flange (in.) in Chapter 10.
t_p	Thickness of panel zone including doubler plates (in.) in Chapter 10.
t_w	Thickness of web (in.) in Chapter 10.
t_z	Thickness of panel zone (doubler plates not necessarily included) (in.) in Chapter 10.
U	Required strength to resist factored loads, or related internal moments and forces in the "Appendix to Chapter 12."
V	The total design lateral force or shear at the base.
V	Shear on a masonry section due to unfactored loads (lb) in the Appendix to Chapter 12
V_m	Shear strength of masonry (lb) in the "Appendix to Chapter 12."
V_n	Nominal shear strength (lb) in the "Appendix to Chapter 12."
V_n	Nominal shear strength of a member (kips) in Chapter 10.
V_s	Shear strength provided by shear reinforcement (lb) in the "Appendix to Chapter 12."
V_t	The design value of the seismic base shear as determined in Sec. 5.8.
V_u	Required shear strength (lb) in the "Appendix to Chapter 12."
V_u	Required shear strength of a member (kips) in Chapter 10.

V_x — The seismic design shear in Story x as determined in Sec. 4.4 or Sec. 5.8.

V_y — Nominal shear strength of an active link (kips) in Chapter 10.

V_{ya} — Nominal shear strength of a member modified by the axial load magnitude (kips) in Chapter 10.

$\tilde{V}_1$ — 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 — Width of a wood shear panel or diaphragm in Chapter 9.

w_i, w_n, w_x — The portion of W that is located at or assigned to Level i, n, or x, respectively.

w_z — Width of panel zone between column flanges (in.) in Chapter 10.

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

Z — Plastic section modulus (in.^{3}) in Chapter 10.

Z_c — Plastic section modulus of a column (in.^{3}) in Chapter 10.

α — The relative weight density of the structure and the soil as determined in Sec. 6A.2.1.

α Fraction of member force transferred across a particular net section in Chapter 10.

α Angle between diagonal reinforcement and longitudinal axis of the member.

β Ratio of shear demand to shear capacity for the story between Level x and $x - 1$.

$\tilde{\beta}$ 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 Computed deflection for serviceability (in.) in the "Appendix to Chapter 12."

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 Computed deflection for serviceability in the "Appendix to Chapter 12."

Δ_{cr} Computed deflection at cracking moment strength level (in.) in the "Appendix to Chapter 12."

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.

$\bar{\delta}_x$, $\bar{\delta}_{x1}$	The deflection of Level x at the center of the mass at and above Level x, Eq. 6A-11 and 6A-14.
ϵ_{mu}	Maximum usable compressive strain of masonry (in./in.) in the "Appendix to Chapter 12."
θ	The stability coefficient for P-delta effects as determined in Sec. 4.6.2.
κ	The overturning moment reduction factor, Eq. 4-6.
ρ	Ratio of required axial force (P_u) to nominal shear strength (V_y) of a link in Chapter 10.
ρ	Ratio of the area of flexural reinforcement to the cross sectional area of masonry on a plane perpendicular to the longitudinal direction of the reinforcement in the "Appendix to Chapter 12."
ρ_b	Reinforcement ratio producing balanced strain conditions in the "Appendix to Chapter 12."
ρ_h	Ratio of the area of shear reinforcement to the cross sectional area of masonry on a plane perpendicular to the reinforcement in the "Appendix to Chapter 12."
ρ_s	Spiral reinforcement ratio for precast prestressed piles in Sec. 7.5.3.4.
ϕ	The capacity reduction factor.
ϕ	Resistance factor for steel in Chapter 10.
ϕ	Strength reduction factor in Chapters 11 and 12.
ϕ_b	Resistance factor for steel beams in Chapter 10.
ϕ_c	Resistance factor for steel columns in compression in Chapter 10.
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.
ϕ_t	Resistance factor for steel columns in tension in Chapter 10.
ϕ_w	Resistance factor for welds in Chapter 10.
ϕP_n	Nominal strength of a column.
λ	Slenderness parameter in Chapter 10.

λ_p	Limiting slenderness parameter for compact element in Chapter 10.
λ_r	Limiting slenderness parameter for noncompact element in Chapter 10.
Ω	Factor of safety in Chapter 10.

Chapter 3

STRUCTURAL DESIGN REQUIREMENTS

3.1 DESIGN BASIS: The seismic analysis and design procedures to be used in the design of buildings and their components shall be as prescribed in this chapter. The design ground motions can occur along any direction of a building. 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, and 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; if an alternate procedure is used, 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 COEFFICIENT: The value of the site coefficient (S) shall be determined from Table 3.2. In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the soil profile does not fit any of the four types indicated in Table 3.2, a site coefficient (S) of 2.0 shall be used.

The design base shear, story shears, overturning moments, and deflections determined in Chapter 4 or Chapter 5 are permitted to be modified in accordance with the "Appendix to Chapter 6" or other approved procedures that account for the effects of soil-structure interaction.

TABLE 3.2
Site Coefficient

Soil Profile Type	Description	Site Coefficient, S
S_1	A soil profile with either: (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 feet per second or (2) stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.	1.0
S_2	A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
S_3	A soil profile containing 20 to 40 feet in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils.	1.5
S_4	A soil profile characterized by a shear wave velocity of less than 500 feet per second containing more than 40 feet of soft clays or silts.	2.0

3.3 STRUCTURAL FRAMING SYSTEMS: The basic structural framing systems to be used are indicated in Table 3.3. Each type is subdivided by the types of vertical element used to resist lateral seismic forces. The structural system used shall be in accordance with the seismic performance category and height limitations indicated in Table 3.3 The appropriate response modification factor (R) and the deflection amplification factor (C_d) indicated in Table 3.3 shall be used in determining the base shear and design story drift. Structural framing and resisting systems that are not contained in Table 3.3 shall be permitted if analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 3.3 for equivalent response modification factor (R) values. Special framing requirements are indicated in Sec. 3.6 and in Chapters 9, 10, 11, and 12 for buildings assigned to the various seismic performance categories.

TABLE 3.3
Structural Systems

Basic Structural System and Seismic Force Resisting System	Response Modification Coefficient, R^a	Deflection Amplification Factor, $C_d{}^b$	Structural System Limitations and Building Height (feet) Limitations[c] — Seismic Performance Category: A & B	C	D[d]	E[e]
<u>Bearing Wall System</u>						
Light frame walls with shear panels	6½	4	NL	NL	160	100
Reinforced concrete shear walls	4½	4	NL	NL	160	100
Reinforced masonry shear walls	3½	3	NL	NL	160	100
Concentrically braced frames	4	3½	NL	NL	160	100
Unreinforced masonry shear walls	1¼	1¼	NL	*f*	NP	NP
<u>Building Frame System</u>						
Eccentrically braced frames, moment resisting connections at columns away from link	8	4	NL	NL	160	100
Eccentrically braced frames, non-moment resisting connections at columns away from link	7	4	NL	NL	160	100
Light frame walls with shear panels	7	4½	NL	NL	160	100
Concentrically braced frames	5	4½	NL	NL	160	*g*
Reinforced concrete shear walls	5½	5	NL	NL	160	100
Reinforced masonry shear walls	4½	4	NL	NL	160	100
Unreinforced masonry shear walls	1½	1½	NL	*f*	NP	NP
<u>Moment Resisting Frame System</u>						
Special moment frames of steel	8	5½	NL	NL	NL	NL
Special moment frames of reinforced concrete	8	5½	NL	NL	NL	NL
Intermediate moment frames of reinforced concrete	4	3½	NL	NL	NP	NP
Ordinary moment frames of steel	4½	4	NL	NL	160	100
Ordinary moment frames of reinforced concrete	2	2	NL	NP	NP	NP
<u>Dual System with a Special Moment Frame Capable of Resisting at Least 25% of Prescribed Seismic Forces</u>						
Eccentrically braced frames, moment resisting connections at columns away from link	8	4	NL	NL	NL	NL
Eccentrically braced frames, non-moment resisting connections at columns away from link	7	4	NL	NL	NL	NL
Concentrically braced frames	6	5	NL	NL	NL	NL
Reinforced concrete shear walls	8	6½	NL	NL	NL	NL
Reinforced masonry shear walls	6½	5½	NL	NL	NL	NL
Wood sheathed shear panels	8	5	NL	NL	NL	NL
<u>Dual System with an Intermediate Moment Frame of Reinforced Concrete or an Ordinary Moment Frame of Steel Capable of Resisting at Least 25% of Prescribed Seismic Forces</u>						
Concentrically braced frames	5	4½	NL	NL	160	100
Reinforced concrete shear walls	6	5	NL	NL	160	100
Reinforced masonry shear walls	5	4½	NL	NL	160	100
Wood sheathed shear panels	7	4½	NL	NL	160	100

TABLE 3.3
Structural Systems

Basic Structural System and Seismic Force Resisting System	Response Modification Coefficient, R [a]	Deflection Amplification Factor, C_d [b]	Structural System Limitations and Building Height (feet) Limitations[c] — Seismic Performance Category: A & B	C	D[d]	E[e]
Inverted Pendulum Structures--Seismic Force Resisting System						
Special moment frames of structural steel	2½	2½	NL	NL	NL	NL
Special moment frames of reinforced concrete	2½	2½	NL	NL	NL	NL
Ordinary moment frames of structural steel	1¼	1¼	NL	NL	NP	NP

[a] Response modification coefficient, R, for use in Sec. 3.5.5, 4.2.1, 5.5, and in other sections throughout the *Provisions*.

[b] Deflection amplification factor, C_d, for use in Sec. 4.6.1 and 4.6.2.

[c] NL = Not Limited and NP = Not Permitted.

[d] See Sec. 3.3.4.1 for a descriptions of building systems limited to buildings with a height of 240 feet or less.

[e] See Sec. 3.3.5 for building systems limited to buildings with a height of 160 feet or less.

[f] The masonry shear walls shall have nominal reinforcement as required by Ref. 12.1, Sec. A.3 (ACI-ASCE 530).

[g] A concentrically braced frame in buildings more than one story in height must be part of a dual system; see Sec. 10.6.1.

3.3.1 DUAL SYSTEM: For a dual system, the moment frame shall be capable of resisting at least 25 percent of the design forces. The total seismic force resistance is to be provided by the combination of the moment frame and the shear walls or braced frames in proportion to their rigidities.

3.3.2 COMBINATIONS OF FRAMING SYSTEMS: Different structural framing systems are permitted along the two orthogonal axes of the building. Combinations of framing systems shall comply with the requirements of this section.

3.3.2.1 Combination Framing Factor: The response modification factor, R, in the direction under consideration at any story shall not exceed the lowest response modification factor (R) for the seismic force resisting system in the same direction considered above that story.

> **EXCEPTION:** Supported structural systems with a weight equal to or less than 10 percent of the weight of the building.

3.3.2.2 Combination Framing Detailing Requirements: The detailing requirements of Sec. 3.6 required by the higher response modification factor (R) shall be used for structural components common to systems having different response modification factors.

3.3.3 SEISMIC PERFORMANCE CATEGORIES A, B, AND C: The structural framing system for buildings assigned to Seismic Performance Categories A, B, and C shall comply with the building height and structural limitations in Table 3.3.

3.3.4 SEISMIC PERFORMANCE CATEGORY D: The structural framing system for a building assigned to Seismic Performance Category D shall comply with Sec. 3.3.3 and the additional provisions of this section.

3.3.4.1 Limited Building Height: The height limits in Table 3.3 may be increased to 240 feet in buildings that have steel braced frames or concrete cast-in-place shear walls. Such buildings shall have braced frames or shear walls arranged in one plane such that they resist no more than the following portion of the seismic forces in each direction including torsional effects:

1. Sixty percent when the braced frame or shear walls are arranged only on the perimeter,

2. Forty percent when some of the braced frames or shear walls are arranged on the perimeter,

3. Thirty percent for other arrangements.

3.3.4.2 Interaction Effects: Moment resisting frames that are enclosed or adjoined by more rigid elements not considered to be part of the seismic force 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 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 designed 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 (also see Sec. 3.8).

3.3.4.4 Special Moment Frames: A special moment frame that is used but not required by Table 3.3 is permitted to be discontinued and supported by a more rigid system with a lower response modification factor (R) provided the requirements of Sec. 3.6.2.4 and 3.6.4.2 are met. Where a special moment frame is required by Table 3.3, the frame shall be continuous to the foundation.

3.3.5 SEISMIC PERFORMANCE CATEGORY E: The framing systems of buildings assigned to Category E shall conform to the requirements of Sec. 3.3.4 for Category D and to the additional requirements and limitations of this section. The height limitation of Sec. 3.3.4.1 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: Buildings shall be classified as regular or irregular based on the plan and vertical configuration.

3.4.1 PLAN IRREGULARITY: Buildings having one or more of the features listed in Table 3.4.1 shall be designated as having plan structural irregularity and shall comply with the requirements in the sections referenced in Table 3.4.1.

3.4.2 VERTICAL IRREGULARITY: Buildings having one or more of the features listed in Table 3.4.2 shall be designated as having vertical irregularity and shall comply with the requirements in the sections referenced in Table 3.4.2.

EXCEPTIONS:

1. Structural irregularities of Types 1 or 2 in Table 3.4.2 do not apply where no story drift ratio under design lateral load is less than or equal to 130 percent of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story drift ratio relationship for the top 2 stories of the building are not required to be evaluated.

2. Irregularities Types 1 and 2 of Table 3.4.2 are not required to be considered for 1- and 2-story buildings.

TABLE 3.4.1
Plan Structural Irregularities

	Irregularity Type and Description	Reference Section	Seismic Performance Category Application
1	Torsional Irregularity--to be considered when diaphragms are rigid in relation to the vertical structural elements that resist the lateral seismic forces. 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.	3.6.4.2 4.4.1	D and E C, D, and E
2	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.	3.6.4.2	D and E
3	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 diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	3.6.4.2	D and E
4	Out-of-Plane Offsets Discontinuities in a lateral force resistance path, such as out-of-plane offsets of the vertical elements.	3.6.4.2	D and E
5	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.	3.6.3.1	C, D, and E

TABLE 3.4.2
Vertical Structural Irregularities

	Irregularity Type and Description	Reference Section	Seismic Performance Category Application
1	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	D and E
2	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	D and E
3	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 and E
4	In-Plane Discontinuity in Vertical Lateral Force Resisting Elements An in-plane offset of the lateral force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting element in the story below.	3.6.4.2	D and E
5	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.	3.6.2.4 3.6.4.2	B, C, D, and E

3.5 ANALYSIS PROCEDURES: A structural analysis shall be made for all buildings in accordance with the requirements of this section. This section prescribes the minimum analysis procedure to be followed. Use of the procedure in Chapter 5 or an alternate generally accepted procedure, including the use of an approved site-specific spectrum, is permitted for any building if approved by the regulatory authority. The limitations on the base shear stated in Chapter 5 apply to dynamic modal analysis.

3.5.1 SEISMIC PERFORMANCE CATEGORY A: Regular or irregular buildings assigned to Category A are not required to be analyzed for seismic forces for the building as a whole. The provisions of Sec. 3.6.1 apply.

3.5.2 SEISMIC PERFORMANCE CATEGORIES B AND C: The analysis procedures in Chapter 4 shall be used for regular or irregular buildings assigned to Category B or C or a more rigorous analysis may be made.

3.5.3 SEISMIC PERFORMANCE CATEGORIES D AND E: The analysis procedures identified in Table 3.5.3 shall be used for buildings assigned to Categories D and E or a more rigorous analysis may be made.

TABLE 3.5.3
Analysis Procedures for Seismic Performance Categories D and E

Building Description	Reference and Procedures
1. Buildings designated as regular up to 240 feet	Chapter 4
2. Buildings that have only vertical irregularities of Type 1, 2, or 3 in Table 3.4.2 and have a height exceeding 5 stories or 65 feet and all buildings exceeding 240 feet in height	Chapter 5
3. All other buildings designated as having plan or vertical irregularities	Chapter 4 and dynamic characteristics shall be given special consideration
4. Buildings in Seismic Hazard Exposure Groups II and III in areas with A_a greater than 0.40 within 6 miles (10 kilometers) of faults having the capability of generating magnitude 7 or greater earthquakes	A site-specific response spectrum shall be used but the design base shear shall not be less than that determined from Sec. 4.2
5. Buildings in areas with A_v of 0.2 and greater with a period of 0.7 seconds or greater located on Type S_4 soils	A site-specific response spectrum shall be used but the design base shear shall not be less than that determined from Sec. 4.2; also, the modal seismic design coefficient, C_{sm}, shall not be limited per Sec. 5.5

3.6 DESIGN, DETAILING REQUIREMENTS, AND STRUCTURAL COMPONENT LOAD EFFECTS: The design and detailing of the components of the seismic force resisting system shall comply with the requirements of this section. Foundation design shall conform to the applicable requirements of Chapter 7. The materials and the systems composed of those materials shall conform to the requirements and limitations of Chapters 9 through 12 for the applicable category.

3.6.1 SEISMIC PERFORMANCE CATEGORY A: The design and detailing of buildings assigned to Category A shall comply with the requirements of this section.

3.6.1.1 Connections: 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. Any smaller portion of the building shall be tied to the remainder of the building with elements having a strength of 1/3 of the effective peak velocity-related acceleration (A_v) times the weight of the smaller portion or 5 percent of the portion's weight, whichever is greater.

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

3.6.1.2 Anchorage of Concrete or Masonry Walls: 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 the greater of a seismic lateral force (F_p) induced by the wall or 1,000 times the effective peak velocity-related acceleration (A_v) pounds per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

3.6.1.3 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.

3.6.2 SEISMIC PERFORMANCE CATEGORY B: Buildings assigned to Category B shall conform to the requirements of Sec. 3.6.1 for Category A and the requirements of this section.

3.6.2.1 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 effects of the combination of loads shall be considered as prescribed in Sec. 3.7. 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.

3.6.2.2 Openings: Where openings occur in shear walls, diaphragms or other plate-type elements, reinforcement at the edges of the openings shall be designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement.

3.6.2.3 Orthogonal Effects: The design seismic forces may be applied separately in each of two orthogonal directions and orthogonal effects may be neglected.

3.6.2.4 Discontinuities in Vertical System: Buildings with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 3.4.2, 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 75 percent of the deflection amplification factor (C_d) times the design force prescribed in Chapter 4.

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

3.6.2.6 Collector Elements: 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.

3.6.2.7 Diaphragms: The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

Floor and roof diaphragms shall be designed to resist the following seismic forces: A minimum force equal to 50 percent the effective peak velocity-related acceleration (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 force 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. Diaphragm connections shall be positive, mechanical type connections.

3.6.2.8 Bearing Walls: Exterior and interior bearing walls and their anchorage shall be designed for a force equal to the effective peak velocity-related acceleration (A_v) times the weight of wall (W_c), normal to the surface, with a minimum force of 10 percent of the weight of the wall. Interconnection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

3.6.2.9 Inverted Pendulum-Type Structures: 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 Chapter 4 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

3.6.3 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C shall conform to the requirements of Sec. 3.6.2 for Category B and to the requirements of this section.

3.6.3.1 Plan Irregularity: Buildings that have plan structural irregularity Type 5 in Table 3.4.1 shall be analyzed for seismic forces applied in the direction that causes the critical load effect. As an alternative, the building may be analyzed independently in any two orthogonal directions and the critical load effect due to direction of application of seismic forces on the building may be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction; the combination requiring the maximum component strength shall be used.

3.6.4 SEISMIC PERFORMANCE CATEGORIES D AND E: Buildings assigned to Category D or E shall conform to the requirements of Sec. 3.6.3 for Category C and to the requirements of this section.

3.6.4.1 Orthogonal Load Effects: Buildings shall be designed for the critical load effect due to application of seismic forces. The alternative procedure in Sec. 3.6.3.1 may be used.

3.6.4.2 Plan or Vertical Irregularities: The design 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.

For buildings having a plan structural irregularity of Type 1, 2, 3, or 4 in Table 3.4.1 or a vertical structural irregularity of Type 4 in Table 3.4.2, 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.

3.6.4.3 Vertical Seismic Forces: The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. Horizontal prestressed components shall be designed for the load combination given by Eq. 3-2a in Sec. 3.7. Horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Sec. 3.7.

3.7 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 the combination of load effects given by Eq. 3-1 or, as applicable, Eq. 3-1a, 3-2, or 3-2a.:

$$(1.1 + 0.5A_v)Q_D \pm 1.0Q_E + (1.0Q_L + 0.7Q_S) \qquad (3\text{-}1)$$

$$(0.9 - 0.5A_v)Q_D \pm 1.0Q_E \qquad (3\text{-}2)$$

where

A_v = the coefficient representing effective peak velocity-related acceleration from Sec. 1.4.1.,

Q_D = the effect of dead load,

Q_L = the effect of live load reduced as permitted in Sec. 2.1,

Q_S = the effect of snow load (in areas where the design snow load is less than 30 pounds per square foot, the load factor on Q_s is permitted to be taken as zero; in areas where the design snow load is greater than 30 pounds per square foot and where siting and load duration conditions warrant and when approved by the regulatory agency, the load factor on Q_s is permitted to be reduced to not less than 0.2), and

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

In Eq. 3-1 and 3-1a in areas where the design snow load is less than 30 pounds per square foot, the load factor on Q_S is permitted to be taken as zero. Where conditions warrant and when approved by the regulatory authority, the load factor on Q_S is permitted to be reduced to not less than 0.2.

For columns supporting discontinuous lateral-force-resisting elements, the axial force in the columns shall be computed using the load combinations in Eq. 3-1a and 3-2a.

$$(1.1 + 0.5A_v)Q_D + 1.0Q_L + 0.7Q_S \pm \left(\frac{2R}{5}\right)Q_E \qquad (3\text{-}1a)$$

The axial forces in such columns need not exceed the capacity of other elements of the structure to transfer such loads to the column.

For brittle materials, systems and connections the following load combination also shall be used:

$$(0.9 - 0.5A_v)Q_D \pm \left(\frac{2R}{5}\right)Q_E \qquad (3\text{-}2a)$$

In Eq. 3-1a and 3-2a, the factor $(2R/5)$ shall be greater than or equal to 1.0. In Eq. 3-1, 3-1a, 3-2, and 3-2a, the term $0.5A_v$ may be neglected where A_v is equal to or less than 0.05.

3.8 DEFLECTION AND DRIFT LIMITS: 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 3.8 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. 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. 4.5.

TABLE 3.8
Allowable Story Drift, Δ_a [a]

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

[a] h_{sx} is the story height below Level x.

Chapter 4

EQUIVALENT LATERAL FORCE PROCEDURE

4.1 GENERAL: This chapter provides required minimum standards for the equivalent lateral force procedure of seismic analysis of buildings. For purposes of analysis, the building is considered to be fixed at the base. See Sec. 3.5 for limitations on the use of this procedure.

4.2 SEISMIC BASE SHEAR: The seismic base shear (V) in a given direction shall be determined in accordance with the following equation:

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

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 areas used for storage, a minimum of 25 percent of the floor live load shall be applicable. Floor live load in public garages and open parking structures is not 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 pounds per square foot of floor area, whichever is greater, shall be applicable.

3. Total operating weight of permanent equipment.

4. In areas where the design snow load is less than 30 pounds per square foot, the load factor on Q_S is permitted to be taken as zero. In areas where the design snow load is greater than 30 pounds per square foot and where siting and load duration conditions warrant and when approved by the regulatory agency, the load factor on Q_S is permitted to be reduced to not less than 0.2.

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 in accordance with the following equations:

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

where

A_v = the coefficient representing effective peak velocity-related acceleration from Sec. 1.4.1,

S = the coefficient for the soil profile characteristics of the site in Table 3.2,

R = the response modification factor in Table 3.3, and

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

A soil-structure interaction reduction is permitted when determined using the "Appendix to Chapter 6" or other generally accepted procedures approved by the regulatory agency.

Alternatively, the seismic design coefficient (C_s) need not be greater than the following equation:

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

where:

A_a = the seismic coefficient representing the effective peak acceleration as determined in Sec. 1.4.1 and

R = the response modification factor in Table 3.3.

4.2.2 PERIOD DETERMINATION: The fundamental period of the building (T) in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period (T) shall not exceed the product of the coefficient for upper limit on calculated period (C_a) from Table 4.2.2 and the approximate fundamental period (T_a) determined from the appropriate requirements of Sec. 4.2.2.1.

TABLE 4.2.2
Coefficient for Upper Limit on Calculated Period

Peak Velocity-Related Acceleration (A_v)	Coefficient 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

4.2.2.1 Approximate Fundamental Period for Concrete and Steel Moment Resisting Frame Buildings: The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

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

where

C_T = 0.035 for moment resisting frame systems of steel in which the frames resist 100 percent of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces,

C_T = 0.030 for moment resisting frame systems of reinforced concrete in which the frames resist 100 percent of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces,

C_T = 0.030 for eccentrically braced steel frames,

C_T = 0.020 for all other building systems, and

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

Alternately, the approximate fundamental period (T_a), in seconds, shall be determined from the following equation for concrete and steel moment resisting frame buildings not exceeding 12 stories in height and having a minimum story height of 10 feet:

$$T_a = 0.1N \tag{4-4a}$$

where N = number of stories.

4.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES: The lateral force (F_x) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \tag{4-5}$$

and

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

where:

C_{vx}	=	vertical distribution factor,
V	=	total design lateral force or shear at the base of the building,
w_i and w_x	=	the portion of the total gravity load of the building (W) located or assigned to Level i or x,
h_i and h_x	=	the height (feet) from 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 shall be 2 or shall 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 equation:

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

where F_i = the portion of the seismic base shear (V) induced at Level i.

The seismic design story shear (V_x) shall be distributed to the various vertical elements of the seismic force resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical resisting elements and the diaphragm.

4.4.1 TORSION: The design shall include the torsional moment (M_t) resulting from the location of the building masses plus the accidental 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.

Buildings of Seismic Performance Categories C, D, and E, where Type 1 torsional irregularity exists as defined in Table 3.4.1 shall have the effects accounted for by increasing the accidental torsion at each level by a torsional amplification factor (A_x) determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}} \right)^2 \tag{4-8}$$

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 torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

4.5 OVERTURNING: The 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 force resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moments at Level x (M_x) shall be determined from the following equation:

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

where:

F_i = the portion of the seismic base shear (V) induced at Level i,

h_i and h_x = the height (in feet) from the base to Level i or x,

τ = 1.0 for the top 10 stories,

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

τ = 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-type structures, shall be designed for the foundation overturning design moment (M_f) at the foundation-soil interface determined using the equation for the overturning moment at Level x (M_x) above with an overturning moment reduction factor (τ) of 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 at the top and bottom of the story under consideration. The deflections of Level x at the center of the mass (δ_x) shall be determined in accordance with following equation:

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

where:

C_d = the deflection amplification factor in Table 3.3 and

δ_{xe} = the deflections determined by an elastic analysis.

The elastic analysis of the seismic force resisting system shall be made using the prescribed seismic design forces of Sec. 3.4.2.

For determining compliance with the story drift limitation of Sec. 3.7, the deflections of Level x at the center of mass (δ_x) shall be calculated as required in this section. 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, the design story drift (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 are not required to be considered when the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{4-11}$$

where:

P_x = the total vertical design load at and above Level x. When calculating the vertical design load for purposes of determining *P*-delta, the individual load factors need not exceed 1.0,

Δ = the design story drift occuring simultaneously with V_x,

V_x = the seismic shear force acting between Level x and x - 1,

h_{sx} = the story height below Level x, and

C_d = the deflection amplification factor in Table 3.3.

The stability coefficient (θ) shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{4-12}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may be conservatively taken as 1.0.

When the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to *P*-delta effects (a_d) shall be determined by rational analysis (see Part 2, *Commentary*). To obtain the story drift for including the *P*-delta effect, the design story drift determined in Sec. 4.6.1 shall be multiplied by 1.0/(1 - θ).

When θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

Chapter 5

MODAL ANALYSIS PROCEDURE

5.1 GENERAL: This chapter provides required standards for the modal analysis procedure of seismic analysis of buildings. See Sec. 3.5 for requirements for use of this procedure. 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 shall 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. The number of modes shall equal the number of stories for buildings less then 3 stories in height.

5.4 PERIODS: The required periods and mode shapes of the building in the direction under consideration shall be calculated by established methods of structural analysis for the fixed base condition using the masses and elastic stiffnesses of the seismic force 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 equations:

$$V_m = C_{sm} W_m \tag{5-1}$$

$$\overline{W_m} = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \tag{5-2}$$

where:

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

W_m = the effective modal gravity load,

w_i = the portion of the total gravity load of the building at Level i, and

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 equation:

$$C_{sm} = \frac{1.2 A_v S}{R T_m^{2/3}} \tag{5-3}$$

where:

A_v = the seismic coefficient representing the effective peak velocity-related acceleration as determined in Sec. 1.4.1,

S = the coefficient for the soil profile characteristics of the site as determined by Table 3.1,

R = the response modification factor determined from Table 3.3, and

T_m = the modal period of vibration (in seconds) of the m^{th} mode of the building.

The modal seismic design coefficient, C_{sm}, is not required to exceed 2.5 times the seismic coefficient representing the effective peak acceleration (A_a) divided by the response modification factor (R).

EXCEPTIONS:

1. The limiting value is not applicable to Category D and E buildings with a period of 0.7 seconds or greater located on Type S_4 soils.

2. For buildings on sites soil profile characteristics S_3 or S_4, the modal seismic design coefficient, C_{sm}, for modes other than the fundamental mode that have periods less than 0.3 seconds is permitted to be determined by the following equation:

$$C_{sm} = \frac{A_a}{R}(1.0 + 5.0 T_m) \tag{5-3a}$$

3. For buildings where any modal period of vibration (T_m) exceeds 4.0 seconds, the modal seismic design coefficient (C_{sm}) for that mode is permitted to be determined by the following equation:

$$C_{sm} = \frac{3 A_v S}{R T_m^{4/3}} \tag{5-3b}$$

where:

A_a = the seismic coefficient representing the effective peak acceleration as determined in Sec. 1.4.1,

A_v = the seismic coefficient representing the effective peak velocity-related acceleration as determined in Sec. 1.4.1,

R = the response modification factor determined from Table 3.3,

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the building, and

S = the coefficient for the soil profile characteristics of the site as determined by Table 3.1.

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 by the following equations:

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

and

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

where:

C_{vsm} = the vertical factor in the m^{th} mode,

V_m = the total design lateral force or shear at the base in the m^{th} mode,

w_i and w_x = the portion of the total gravity load of the building (W) located or assigned to Level i or x,

ϕ_{xm} = the displacement amplitude at the x^{th} level of the building when vibrating in its m^{th} mode, and

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

The modal deflection at each level (δ_{xm}) shall be determined by the following equations:

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

and

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

where:

C_d = the deflection amplification factor determined from Table 3.3,

δ_{xem} = the deflection of Level x in the m^{th} mode at the center of the mass at Level x determined by an elastic analysis,

g = the acceleration due to gravity (feet per second squared),

T_m = the modal period of vibration, in seconds, of the m^{th} mode of the building,

F_{xm} = the portion of the seismic base shear in the m^{th} mode, induced at Level x, and

w_x = the portion of the total gravity load of the building (W) located or assigned to Level x.

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 the appropriate equation in Sec. 5.6 shall be computed for each mode by linear static methods.

5.8 DESIGN VALUES: The design value for the modal base shear (V_t); each of the story shear, moment and drift quantities; and the deflection at each level shall be determined by combining their modal values as 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 or by the complete quadratic combination (CQC) technique..

The base shear (V) using the equivalent lateral force procedure in Chapter 4 shall be calculated using a fundamental period of the building (T), in seconds, of 1.2 times the coefficient for upper limit on the calculated period (C_a) times the approximate fundamental period of the building (T_a). Where the design value for the modal base shear (V_t) is less than the calculated base shear (V) using the equivalent lateral force procedure, the design story shears, moments, drifts and floor deflections shall be multiplied by the following modification factor:

$$\frac{V}{V_t} \tag{5-7}$$

where:

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Chapter 4 and

V_t = the modal base shear, calculated in accordance with this section.

The modal base shear (V_t) is not required to exceed the base shear from the equivalent lateral force procedure in Chapter 4.

EXCEPTION: For buildings in areas with an effective peak velocity related (A_v) value of 0.2 and greater with a period of 0.7 second or greater located on Type S_4 soil, the design base shear shall not be less than that determined using the equivalent lateral force procedure in Chapter 4 (see Sec. 3.5.3).

5.9 HORIZONTAL SHEAR DISTRIBUTION: The distribution of horizontal shear shall be in accordance with the requirements of Sec. 4.4.

5.10 FOUNDATION OVERTURNING: The foundation overturning moment at the foundation-soil interface shall be permitted to be reduced by 10 percent.

5.11 P-DELTA EFFECTS: The *P*-delta effects shall be determined in accordance with Sec. 4.6.2. The story drifts and story shears shall be determined in accordance with Sec. 4.6.1.

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:

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

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

$\overline{W}$ = the effective gravity load of the building, which shall be taken as $0.7W$, 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 $0.7V$.

6A.2.1.1 Effective Building Period: The effective period ($\tilde{T}$) shall be determined as follows:

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

where:

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

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

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

$\bar{h}$ = 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 the *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.2.1.1 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 = $\gamma v^2_{so}/g$ = 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.2.1.1
Values of G/G_0 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}	0.09	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:

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

where:

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

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

r_a and r_m = characteristic foundation lengths defined by:

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

and

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

where

A_o = the area of the foundation and

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:

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

where:

β_o = the foundation damping factor as specified in Figure 6A.2.1.2.

The values of β_o corresponding to A_v = 0.15 in Figure 6A.2.1.2 shall be determined by averaging the results obtained from the solid lines and the dashed lines.

The quantity r in Figure 6A.2.1.2 is a characteristic foundation length that shall be determined as follows:

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

$$r = r_a = \sqrt{\frac{A_o}{\pi}}$$

For $\bar{h}/L_o \geq 1$,

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

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. A6-9 shall be replaced by:

$$\beta'_o = \left(\frac{4D_s}{V_s\tilde{T}}\right)^2 \beta_o \tag{6A-10}$$

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.

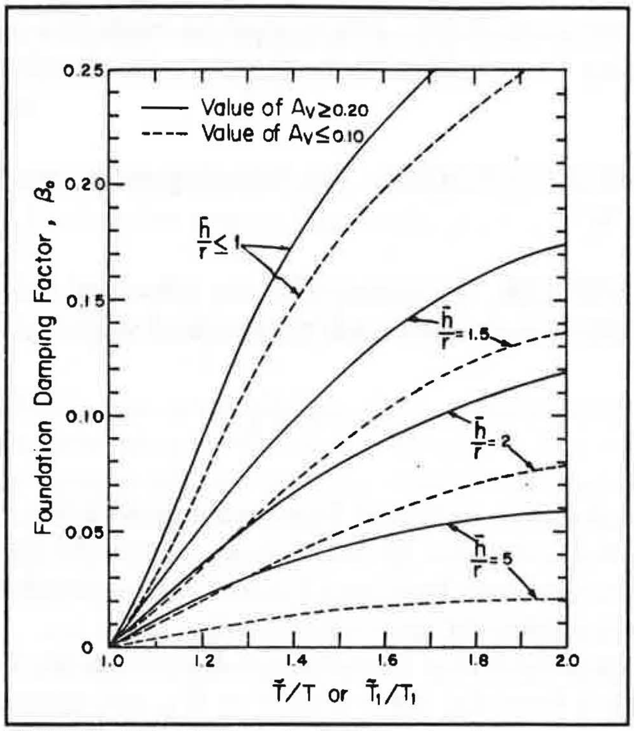

FIGURE 6A.2.1.2 Foundation damping factor.

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:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V\left(\dfrac{M_o h_x}{K_\theta} + \delta_x\right)} \tag{6A-11}$$

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 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 k from Eq. 6A-4 with $W = W_1$, and computing h as follows:

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

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_0 from Figure 6A.2.1.2. No reduction shall be made in the shear components contributed by the higher modes of vibration. The reduced base shear (V_1) shall in no case be taken less than $0.7V_1$.

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 (V_1) instead of V_1. The modified modal deflections ($\tilde{\delta}_x m$) shall be determined as follows:

$$\tilde{\delta}_{xl} = \frac{\tilde{V}_1}{V_1\left(\dfrac{M_{ol} h_x}{K_\theta} + \delta_{xl}\right)} \tag{6A-14}$$

and

$$\tilde{\delta}_{xm} = \delta_x \tag{6A-15}$$

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 assigned to Category A or B.

7.4 SEISMIC PERFORMANCE CATEGORY C: Foundations for 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.

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 lateral spreading, 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 forces 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 the effective peak velocity-related acceleration (A_v) divided by 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 Ref. 11.1. 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 region.

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. 11.6.2.

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 assigned to 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 owner shall submit to the regulatory agency a written report that includes an evaluation of potential site hazards such as slope instability, liquefaction, and surface rupture due to faulting or lateral spreading and 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 for Soil Profile Type S_1 in Table 3.3, 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. 10.10 or 11.3.3 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 $0.20M_{nb}$ (where M_{nb} is the unfactored ultimate moment capacity at balanced strain conditions as defined in Ref. 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: This chapter establishes minimum design levels for architectural, mechanical, and electrical systems and components recognizing occupancy use, occupant load, and need for operational continuity

All architectural, mechanical, and electrical systems and components and systems in buildings shall be designed and constructed to resist seismic forces determined in accordance with this chapter.

EXCEPTION: The following components and systems are exempt from the requirements of this chapter:

1. Those systems or components assigned a performance criteria factor of 0.5 in Table 8.2.2 or 8.3.2a in Seismic Hazard Exposure Group I buildings located in areas with a value of A_v less than 0.15 or in Seismic Hazard Exposure Group II buildings located in areas with a value of A_v less than 0.05.

2. Elevator systems in Seismic Hazard Exposure Group I buildings located in areas with a value of A_v less than 0.15 or in Seismic Hazard Exposure Group II buildings located in areas with a value of A_v less than 0.05.

Seismic Hazard Exposure Groups are determined in Sec. 1.4.2. Mixed Occupancy requirements are presented in Sec. 1.4.2.4.

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 shall not cause the failure of an architectural, mechanical, or electrical system or component with a higher performance criteria factor (P). The effect on the response of the structural system and deformational capability of architectural, electrical, and mechanical systems or components shall be considered where these systems or components interact with the structural system.

8.1.1 COMPONENT FORCE APPLICATION: The component seismic force shall be applied at the center of gravity of the component nonconcurrently in any horizontal direction. Mechanical and electrical components and systems shall be designed for an additional vertical force of 33 percent of the horizontal force acting up or down.

8.1.2 COMPONENT FORCE TRANSFER: Components shall be attached such that the component forces are transferred to the structural system of the building. Component seismic attachments shall be positive connections without consideration of frictional resistance.

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

8.2 ARCHITECTURAL COMPONENT DESIGN:

8.2.1 GENERAL: Systems or components listed in Table 8.2.2 and their attachments shall be designed and detailed in accordance with the requirements of this chapter. The design criteria for systems or components shall be included as part of the design documents.

8.2.2 FORCES: Architectural components and their means of attachment shall be designed for seismic forces (F_p) determined in accordance with the following equation:

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

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 effective peak velocity-related acceleration from Sec. 1.4.1,

C_c = the seismic coefficient for architectural components from Table 8.2.2,

P = the performance criteria factor from Table 8.2.2, and

W_c = the weight of the architectural component.

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 building code horizontal loads exceed F_p for interior partitions, these loads shall govern the design.

TABLE 8.2.2
Architectural Component Seismic Coefficient (C_c) and Performance Criteria Factor (P) [a]

Architectural Component	Component Seismic Coefficient (C_c)	Performance Criteria Factor (P)		
		Seismic Hazard Exposure Group		
		I	II	III
Exterior nonbearing walls	0.9	1.5[d]	1.5[b]	1.5
Interior nonbearing walls				
Stair enclosures	1.5	1.0	1.0[c]	1.5
Elevator shaft enclosures	1.5	0.5[e]	0.5[c]	1.5
Other vertical shaft enclosures	0.9	1.0	1.0	1.5
Other nonbearing walls	0.9	1.0	1.0	1.5
Cantilever elements				
Parapets, chimneys, or stacks	3.0	1.5	1.5	1.5
Wall attachments (see Sec. 8.2.3)	3.0	1.5[d]	1.5[b]	1.5
Veneer connections	3.0	0.5	1.0[g]	1.0
Penthouses	0.6	NR	1.0	1.0
Structural fireproofing	0.9	0.5[f]	1.0[c]	1.5
Ceilings				
Fire-rated membrane	0.9	1.0	1.0	1.5
Nonfire-rated membrane	0.6	0.5	1.0	1.0
Storage racks more than 8 feet in height (contents included)	1.5	1.0	1.0	1.5
Access floors (supported equipment included)	2.0	0.5	1.0	1.5
Elevator and counterweight guiderails and supports	1.25	1.0	1.0	1.5
Appendages				
Roofing units	0.6	NR	1.0[b]	1.0
Containers and miscellaneous components (free standing)	1.5	NR	1.0	1.0
Partitions				
Horizontal exits including ceilings	0.9	1.0	1.5	1.5
Public corridors	0.9	0.5	1.0	1.5
Private corridors	0.6	NR	0.5	1.5
Full height area separation partitions	0.9	1.0	1.0	1.5
Full height other partitions	0.6	0.5	0.5	1.5
Partial height partitions	0.6	NR	0.5	1.0

TABLE 8.2.2 continued

NR = Not required.

[a] See Sec. 8.1 for exceptions.

[b] *P* may be reduced by 0.5 if the area facing the exterior wall is normally inaccessible for a distance of 10 feet and the building is only one story.

[c] *P* shall be increased by 0.5 if the building is more than four stories or 40 feet in height.

[d] *P* shall be increased by 0.5 if the area facing the exterior wall is normally accessible within a distance of 10 feet plus 1 foot for each floor height.

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

[f] *P* shall be increased by 0.5 for an occupancy containing flammable gases, liquids, or dust.

[g] *P* may be reduced by 0.5 if the area facing the exterior wall is normally inaccessible for a distance of 10 feet plus 1 foot of each floor of height.

8.2.3 EXTERIOR WALL PANEL CONNECTIONS: The connections of exterior wall panels to the building seismic resisting system shall be designed for the design story drift as determined in Sec. 4.6.1 or in accordance with Sec. 5.6 or 5.8.

8.2.4 ARCHITECTURAL COMPONENT DEFORMATION: Architectural components shall be designed for the design story drift of the structural resisting system as determined in accordance with Sec. 4.6.1 or Sec. 5.8. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members.

EXCEPTION: Architectural components having a performance criteria factor of 0.5 are to be designed for 50 percent of the design story drift.

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 CEILINGS: Provision shall be made for the lateral support and/or interaction of other architectural, mechanical, and electrical systems or components that may be incorporated into the ceiling and may impose seismic forces into the ceiling system.

8.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN:

8.3.1 GENERAL: Systems or components listed in Table 8.3.2a and their attachments shall be designed and detailed in accordance with the requirements of this chapter. The design 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 this section.

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

8.3.2 FORCES: Mechanical and electrical components and systems shall be designed for seismic forces determined in accordance with the following equation:

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

where:

A_v = the seismic coefficient representing effective peak velocity-related acceleration from Sec. 1.4.1,

C_c = the seismic coefficient for mechanical and electrical components from Table 8.3.2a,

P = the performance criteria factor from Table 8.3.2a,

a_c = the amplification factor determined in accordance with Table 8.3.2b, and

W_c = The operating weight of the mechanical or electrical component or system.

Alternatively, the seismic forces (F_p) are to be determined by a properly substantiated dynamic analysis subject to approval by the building code official.

TABLE 8.3.2a
Mechanical and Electrical Component and System Seismic Coefficient (C_c) and Performance Criteria Factor (P) [a]

Mechanical and Electrical Component or System	Component or System Seismic Coefficient $(C_c)^b$	Performance Criteria Factor (P)		
		Seismic Hazard Exposure Group		
		I	II	III
Fire protection equipment and systems	2.0	1.5	1.5	1.5
Emergency or standby electrical systems	2.0	1.5	1.5	1.5
Elevator drive, suspension system, and controller anchorage	1.25	1.0	1.0	1.5
General equipment 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, conduit, and cable trays[c] 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	2.0	0.5	1.0	1.5
Manufacturing and process machinery	0.67	0.5	1.0	1.5
Pipe systems				
Gas and high hazard piping	2.0	1.5	1.5	1.5
Fire suppression piping	2.0	1.5	1.5	1.5
Other pipe systems[d]	0.67	NR	1.0	1.5
HVAC and service ducts[e]	0.67	NR	1.0	1.5
Electrical panel boards and dimmers	0.67	NR	1.0	1.5
Lighting fixtures[f]	0.67	0.5	1.0	1.5
Conveyor systems (nonpersonnel)	0.67	NR	NR	1.5

TABLE 8.3.2a continued

NR = Not required.

[a] See Sec. 8.1 for general exceptions.
[b] C_c values are for horizontal forces; C_c values for vertical forces shall be taken as one-third of the horizontal values.
[c] Seismic restraints are not required for electrical conduit and cable trays for any of the following conditions: (1) conduit and cable trays suspended by individual hangers 12 inches in length from the top of the conduit to the supporting structure, (2) conduit in boiler and mechanical rooms that has less than 1-1/4 inch inside diameter, (3) conduit in other areas that has less than 2-1/2 inch inside diameter.
[d] Seismic restraints are not required for any of the following conditions for other pipe systems: (1) piping suspended by individual hangers 12 inches or less in length from the top of the pipe to the supporting structure, (2) piping in boiler and mechanical rooms that has less than 1-1/4 inch inside diameter, (3) piping in other areas that has less than 2-1/2 inches inside diameter.
[e] Seismic restraints are not required for any one of the following conditions for HVAC or service ducts: (1) ducts suspended by hangers 12 inches or less in length from the top of the duct to the supporting structure, (2) ducts that have a cross sectional area less than 6 square feet.
[f] Pendulum lighting fixtures shall be designed using a Component Seismic Coefficient (C_c) of 1.5. The vertical support shall be designed with a factor of safety of 4.0.

TABLE 8.3.2b
Attachment Amplification Factor (a_c)

Component Supporting Mechanism	Attachment Amplification Factor (a_c)
Fixed or direct connection	1.0
Seismic activated restraining device	1.0
Resilient support system where[a]:	
$T_{c/T} < 0.6$ or $T_{c/T} > 1.4$	1.0
$T_{c/T} \geq 0.6$ or $T_{c/T} \leq 1.4$	2.0

[a] T is the fundamental period of the building in seconds determined by Sec. 4.2.2 or Sec. 5.4. T_c is the fundamental period in seconds of the component and its means of attachment determined by Sec. 8.3.3.

TABLE 8.3.2a
Mechanical and Electrical Component and System Seismic Coefficient (C_c) and Performance Criteria Factor (P) [a]

Mechanical and Electrical Component or System	Component or System Seismic Coefficient $(C_c)^b$	Performance Criteria Factor (*P*)		
		Seismic Hazard Exposure Group		
		I	II	III
Fire protection equipment and systems	2.0	1.5	1.5	1.5
Emergency or standby electrical systems	2.0	1.5	1.5	1.5
Elevator drive, suspension system, and controller anchorage	1.25	1.0	1.0	1.5
General equipment 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, conduit, and cable trays[c] 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	2.0	0.5	1.0	1.5
Manufacturing and process machinery	0.67	0.5	1.0	1.5
Pipe systems				
Gas and high hazard piping	2.0	1.5	1.5	1.5
Fire suppression piping	2.0	1.5	1.5	1.5
Other pipe systems[d]	0.67	NR	1.0	1.5
HVAC and service ducts[e]	0.67	NR	1.0	1.5
Electrical panel boards and dimmers	0.67	NR	1.0	1.5
Lighting fixtures[f]	0.67	0.5	1.0	1.5
Conveyor systems (nonpersonnel)	0.67	NR	NR	1.5

TABLE 8.3.2a continued

NR = Not required.

[a] See Sec. 8.1 for general exceptions.

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

[c] Seismic restraints are not required for electrical conduit and cable trays for any of the following conditions: (1) conduit and cable trays suspended by individual hangers 12 inches in length from the top of the conduit to the supporting structure, (2) conduit in boiler and mechanical rooms that has less than 1-1/4 inch inside diameter, (3) conduit in other areas that has less than 2-1/2 inch inside diameter.

[d] Seismic restraints are not required for any of the following conditions for other pipe systems: (1) piping suspended by individual hangers 12 inches or less in length from the top of the pipe to the supporting structure, (2) piping in boiler and mechanical rooms that has less than 1-1/4 inch inside diameter, (3) piping in other areas that has less than 2-1/2 inches inside diameter.

[e] Seismic restraints are not required for any one of the following conditions for HVAC or service ducts: (1) ducts suspended by hangers 12 inches or less in length from the top of the duct to the supporting structure, (2) ducts that have a cross sectional area less than 6 square feet.

[f] Pendulum lighting fixtures shall be designed using a Component Seismic Coefficient (C_c) of 1.5. The vertical support shall be designed with a factor of safety of 4.0.

TABLE 8.3.2b
Attachment Amplification Factor (a_c)

Component Supporting Mechanism	Attachment Amplification Factor (a_c)
Fixed or direct connection	1.0
Seismic activated restraining device	1.0
Resilient support system where[a]:	
$T_{c/T} < 0.6$ or $T_{c/T} > 1.4$	1.0
$T_{c/T} \geq 0.6$ or $T_{c/T} \leq 1.4$	2.0

[a] T is the fundamental period of the building in seconds determined by Sec. 4.2.2 or Sec. 5.4. T_c is the fundamental period in seconds of the component and its means of attachment determined by Sec. 8.3.3.

8.3.3 COMPONENT PERIOD: The fundamental period of the component and its means of attachment to the building, T_c, in seconds shall be determined by the following equation:

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

where:

W_c = the weight of the component (lb) and

K = stiffness of the resilient support system determined in terms of load per unit deflection at the center of gravity of the component.

Alternatively, the fundamental period of the component in seconds (T_c) is to be determined by experimental test data or by a properly substantiated analysis.

8.3.4 COMPONENT ATTACHMENT: Component supporting mechanisms 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 means of attachment.

Systems, components, and the means of their attachment shall be designed to accommodate relative seismic displacements between points of support. Displacements at points of support shall be determined in accordance with Eq. 4-10. Relative lateral displacements at points of support shall be determined considering the difference in elevation between the supports and considering full out-of-phase displacements across portions of buildings that may move in a differential manner such as at seismic and expansion joints.

8.3.5 COMPONENT CERTIFICATION: When the direct attachment is used for components with performance criteria factors (P) of 1.0 or greater in buildings assigned an effective peak velocity-related acceleration (A_v) equal to or greater than 0.15 as determined from Sec. 1.4.1, the manufacturers certification of the component seismic acceleration operational capacity which meets the requirements of this section shall be submitted to the regulatory agency.

8.3.6 UTILITY AND SERVICE INTERFACES:

8.3.6.1 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 an effective peak velocity-related acceleration (A_v) equal to 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 times the effective peak acceleration (A_a).

8.3.6.2 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.

8.3.7 SITE-SPECIFIC CONSIDERATIONS: The possible interruption of utility service shall be considered in relation to designated seismic systems in Seismic Hazard Exposure Group III as defined in Sec. 1.4.2. Specific attention shall be given to the vulnerability of underground utilities from the service connection in areas of S_3 or S_4 soils where the effective peak velocity-related acceleration coefficient (A_v) is equal to or greater than 0.15.

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 A17.1-1984, *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 8.2.2. W_c is defined as follows:

Element = W_c,

Traction Car = $C + 0.4L$,

Counterweight = W, and

Hydraulic = $C + 0.4L + 0.25P$,

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, Table 8.3.2a, and Table 8.3.2b.

8.4.4 SEISMIC CONTROLS: All elevators with a speed of 150 feet per minute 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 device 1 or a combination of signals from devices 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	NFoPA T901 (1986)
Ref. 9.2	American Softwood Lumber Standard	PS 20-70 (1986)
Ref. 9.3	Methods for Establishing Structural Grades and Allowable Properties for Visually Graded Lumber	ASTM D245 (1988)
Ref. 9.4	Methods for Establishing Clear Wood Strength Values	ASTM D2555 (1988)
Ref. 9.5	Softwood Plywood--Construction and Industrial	PS 1-83 (1983)
Ref. 9.6	Wood Particleboard	ANSI A208.1 (1989)
Ref. 9.7	Preservative Treatment by Pressure Process	AWPA C1(1988), C2 and C3 (1989), C9 (1985), and C28 (1989)
	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 of Standard Specification for Structural Glued Laminated Timber of Softwood Species	AITC 117 (1989)
Ref. 9.10	Wood Poles	ANSI 05.1 (1988)
Ref. 9.11	Round Timber Piles	ASTM D25 (1988a)

*Chapter 9 was rewritten for the 1991 Edition by the Wood Technical Subcommittee and the revised chapter was approved by the required two-thirds majority of the BSSC members during balloting. However, several members raised issues (concerning, for example, the requirements for braced wall panels for conventional construction, clarity of presentation of the requirements, and the need for cross-referencing within the chapter) that will be considered during the next updating of the *Provisions*.

Ref. 9.12	One- and Two-Family Dwelling Code	Council of American Building Officials (CABO), 1989
Ref. 9.13	Gypsum Wallboard	ASTM C36-84
Ref. 9.14	Fiberboard Nail-Base Sheathing	ASTM D2277-87
Ref. 9.15	Plywood Design Specifications	APA (1986)
Ref. 9.16	Diaphragms	APA (1987)
Ref. 9.17	Performance Policies and Standards for Structural Use Panels	APA PRP-108 (1988)
Ref. 9.18	Design Capacities of APA Performance-Rated Structural Use Panels	APA N375 (1988)
Ref. 9.19	Span Tables for Joists and Rafters	NFoPA T903 (1977)

Dimensions for wood products and associated products designated in this section are nominal dimensions and actual dimensions shall be not less than prescribed by the reference standards. For diaphragms and shear walls, the acceptable types of sheathing listed in Sec. 9.8.3, except Sec. 9.8.3.1, 9.8.3.4 and 9.8.3.7, shall have nominal sheet sizes of 4 feet by 8 feet or larger.

9.2 STRENGTH OF MEMBERS AND CONNECTIONS: The design strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor (ϕ) and 2.0 times the working stresses permitted in the reference documents and in this chapter. The value of the capacity reduction factor (ϕ) shall be as shown in Table 9.2.

TABLE 9.2
Capacity Reduction Factors

Member or Connection	Capacity Reduction Factor (ϕ)
All stresses in wood members	1.0
Bolts and other timber connectors not listed below	1.0
Shear on carriage bolts not having washers under the head	0.67
Lag screws and wood screws	0.90
Shear on diaphragms and shear walls as given in this chapter	0.85

9.3 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A may be constructed using any of the materials and procedures permitted in the reference documents and need conform only to the requirements of Sec. 3.6.1.

9.4 SEISMIC PERFORMANCE CATEGORY B: Buildings assigned to Category B may be constructed using any of the materials and procedures permitted in the reference documents and this chapter except as limited by this section.

9.4.1 CONSTRUCTION LIMITATIONS: Buildings not over three stories or 40 feet in height, not exempted by Sec. 1.3, shall conform to the provisions of Sec. 9.8 or such buildings and all other buildings shall be designed in conformance with Sec. 9.9.

9.5 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C shall conform to all of the requirements for Category B and to the additional requirements of this section.

9.5.1 MATERIAL LIMITATIONS, STRUCTURAL USE PANEL SHEATHING: Where structural use panel sheathing is used as siding on the exterior of outside walls, it shall be of the exterior type. Where structural use panel sheathing is used elsewhere, it shall be bonded by intermediate or exterior glue.

9.5.2 CONSTRUCTION LIMITATIONS, CONVENTIONAL CONSTRUCTION: Buildings not exempted in Sec. 1.3 that are constructed in conformance with Sec. 9.8 shall be limited to one level, one story, or 20 feet in height.

9.5.3 DETAILING REQUIREMENTS: The construction shall comply with the requirements given below.

9.5.3.1 Anchorage of Concrete or Masonry Walls: For Seismic Hazard Exposure Group III buildings in areas where the value of A_v is equal to or greater than 0.10, the diaphragm sheathing may not be used to provide the ties and splices required by Sec. 3.6.1.1 and 3.6.1.2.

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

9.6 SEISMIC PERFORMANCE CATEGORY D: Buildings assigned to Category D shall conform to all the requirements for Category C and to the additional requirements of this section.

9.6.1 MATERIAL LIMITATIONS, SHEATHING MATERIALS: Fiberboard sheathed walls shall not be considered to be part of the seismic force resisting system.

9.6.2 CONSTRUCTION LIMITATIONS, CONVENTIONAL CONSTRUCTION: Buildings assigned to Seismic Hazard Exposure Groups II and III in Seismic Performance Category D shall conform to Sec. 9.9.

9.6.3 FRAMING SYSTEMS: The limitations on framing systems that may be used in Category D construction are given below. Framing shall be designed in accordance with Sec. 9.9.

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

9.6.3.2 Anchorage of Concrete and Masonry Walls: Ties and splices required in Sec. 3.6.1.1 and 3.6.1.2 shall be provided and the diaphragm sheathing shall not be considered for this purpose.

9.6.4 DETAILING REQUIREMENTS: 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 apply a capacity reduction factor of 0.9.

9.7 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.

9.7.1 MATERIAL LIMITATIONS: Walls sheathed with gypsum sheathing board, particleboard, gypsum wallboard, fiberboard, gypsum plaster, or cement plaster shall not be considered to be part of the seismic force resisting system.

9.7.2 FRAMING SYSTEMS: Framing shall be designed in conformance with Sec. 9.9. Unblocked structural use panel sheathed diaphragms shall not be considered to be part of the seismic force resisting system.

9.7.3 DIAPHRAGM AND SHEAR WALL LIMITATIONS: Structural use panel sheathing used for diaphragms and shear walls that are part of the seismic force resisting system shall be applied directly to the framing members.

> EXCEPTION: Structural use panel sheathing may be used as a diaphragm when fastened over solid lumber planking or laminated decking provided the panel joints and lumber planking or laminated decking joints do not coincide.

The allowable working stress shear for structural use panel sheathed vertical shear walls used to resist seismic forces in buildings with concrete or masonry walls shall be one-half the values set forth in Table 9.10.1.3b.

9.8 CONVENTIONAL LIGHT FRAME CONSTRUCTION: Conventional light frame construction is a system of repetitive horizontal and vertical framing members selected from tables in Ref. 9.19 and conforming to the framing and bracing requirements of Ref. 9.12 except as modified by the provisions in this section. This system is limited, unless exempted by Sec. 1.2 or 1.3, to buildings three stories or 40 feet in height in Category B or to buildings one story or 20 feet in height in Category C and in Seismic Hazard Exposure Group I in Category D.

The gravity dead load of the construction is limited to 15 pounds per square foot for roofs and exterior walls and 10 pounds per square foot for floors and partitions.

EXCEPTION: Masonry veneer may be used on one-story Category B buildings.

9.8.1 BRACED WALLS: The following braced wall requirements shall apply as a minimum.

9.8.1.1 Braced Wall Spacing: Braced exterior walls and braced partitions shall be located at not more than 25 foot intervals in each direction.

EXCEPTION: For Category B buildings, the spacing may be increased to 35 feet.

9.8.1.2 Braced Wall Sheathing Requirements: All braced walls and partitions shall be effectively and thoroughly braced by one of the types of sheathing prescribed in Sec. 9.8.3 for each 25 feet of building length along each braced line. Such bracing shall be distributed along the length of the braced line with sheathing placed at each end of the wall or partition or as near thereto as possible. 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 for diagonal boards or structural use panel sheets and shall be at least 96 inches in width covering six 16-inch stud spaces or four 24-inch stud spaces for all other sheathing. All vertical panel sheathing joints shall occur over studs. Sheathing shall be fastened to all studs and plates. All wall framing to which sheathing used for bracing is applied shall conform to Ref. 9.2 for 2x or larger members.

Panel sheathing nailing shall be the minimum given in Tables 9.10.1.3b, 9.10.1.4a, and 9.10.1.4b. Nailing for diagonal boards shall be as prescribed in Sec. 9.10.1.1. Nailing for particleboard shall be the minimum prescribed for fiberboard in Table 9.10.1.4a. Fasteners shall be placed at least 3/8 inch from ends of boards or edges of sheets.

Cripple stud walls shall be braced as required for braced walls or partitions and shall be considered an additional story. Where interior post and girder framing is used, the sheathing at exterior walls shall be increased to compensate for that which would normally occur at interior braced walls.

9.8.2 WALL FRAMING AND CONNECTIONS: The following wall framing and connection details shall apply as a minimum.

9.8.2.1 Wall Anchorages: Anchorage for wall sills to concrete or masonry foundations conforming to the requirements of Chapters 11 and 12 shall be provided. Such anchorage may be provided by 1/2-inch diameter anchor bolts having a minimum embedment of 7 bolt diameters spaced at not over 6 feet on center for one- and two-story buildings and at not more than 4 feet on center for buildings over two stories in height. Other anchorage devices having equivalent capacity may be used.

9.8.2.2 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 4 feet. Single top plates may be used provided they are spliced by framing devices providing capacity equivalent to the lapped splice prescribed for double top plates.

9.8.2.3 Bottom Plates: Studs shall have full bearing on a plate or sill conforming to Ref. 9.2 for 2x or larger members having a width at least equal to the width of the studs.

9.8.2.4 Roof and Floor to Braced Wall Connection: Provision shall be made to transfer forces from roofs and floors to braced walls and from the braced walls in upper stories to the braced walls in the story below. Such transfer can be accomplished by blocking and nailing or by metal framing devices capable of transmitting the equivalent lateral force.

Roof to braced wall connections for buildings with maximum dimensions not over 50 feet may be made at exterior walls only and larger buildings shall have connections at the exterior walls and interior bearing walls. Floor to braced wall connections shall be made at every braced wall. The connections shall be distributed along the length of the braced wall. Where all wood foundations are used, the transfer force shall be the same as that applicable for the number of stories indicated above.

9.8.3 ACCEPTABLE TYPES OF WALL SHEATHING: Sheathing used for bracing shall conform to one of the following types:

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

9.8.3.2 Structural Use Panel Sheets: Structural use panel sheets 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.

9.8.3.3 Fiberboard Sheets: Fiberboard sheets not less than 7/16 inch thick applied with the long dimension vertical on studs spaced not over 16 inches on center.

9.8.3.4 Gypsum Sheathing Boards: Gypsum sheathing boards not less than 1/2 inch thickness on studs spaced not over 16 inches on center.

9.8.3.5 Particleboard Sheets: Particleboard sheets 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.

9.8.3.6 Gypsum Wallboard Sheets: Gypsum wallboard sheets not less than 1/2 inch thick on studs spaced not over 24 inches on center.

9.8.3.7 Gypsum and Cement Plaster: Gypsum plaster over gypsum sheathing boards and cement plaster on metal lath.

9.9 ENGINEERED WOOD CONSTRUCTION: For buildings for which the conventional construction provisions of Sec. 9.8 cannot be utilized, the design, proportioning, and detailing of wood systems, members, and connections shall be in accordance with the reference documents and this section.

9.9.1 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. Positive connections shall be provided to resist uplift and lateral displacement.

9.9.2 DIAPHRAGM AND SHEAR WALL REQUIREMENTS: Diaphragm and shear wall framing and detailing shall conform to the requirements of this section.

9.9.2.1 Framing: All framing members used for shear panel construction shall conform to Ref. 9.2 for 2x or larger members. Boundary members and chords in diaphragms and shear walls and collectors transferring forces to such elements shall be designed and detailed for the induced axial forces. Boundary members shall be tied together at all corners.

Openings in diaphragms and shear walls shall be designed and detailed to transfer the shear and axial forces induced by the discontinuity created by the opening and the details shall be shown on the approved plans.

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

9.9.2.3 Torsion: Buildings with two lines of resistance and having torsional irregularity due to stiffness ratios between the two lines of resistance greater than 4 to 1 or with one line of resistance in either orthogonal direction shall meet the following requirements:

9.9.2.3.1: Diaphragm sheathing shall conform to Sec. 9.10.1.1 through 9.10.1.3.

9.9.2.3.2: The width of the diaphragm normal to the orthogonal axis about which the torsional irregularity exists shall not exceed 25 feet and the l/w ratio shall not exceed 1:1 for one-story buildings or 1:1.5 for buildings over one story in height where l = the length of a shear panel diaphragm and w = the width of a shear panel or diaphragm.

EXCEPTION: Where calculations demonstrate that the diaphragm deflections can be tolerated, the depth may be increased and the l/w ratio may be increased to 1.5:1 when sheathed in conformance with Sec. 9.10.1.1 or to 2:1 when sheathed in conformance with Sec. 9.10.1.2 or 9.10.1.3.

9.10 DIAPHRAGMS AND SHEAR WALLS: The width of a shear panel in a diaphragm or shear wall shall not be less than 2 feet and the h/w ratio of a shear wall shall not be greater than 2 for unblocked structural use panels and diagonal sheathing and not greater than 3-1/2 for blocked structural use panels and double diagonal sheathing where h = the height of a shear panel or shear wall and w = the width of a shear panel or shear wall.

9.10.1 SHEAR PANEL REQUIREMENTS: Shear panels in diaphragms and shear walls shall conform to the requirements of this section. Fasteners shall be placed at least 3/8 inch from ends of boards or edges of sheets. All vertical panel sheathing joints in shear walls shall occur over studs. Where prescribed in Table 9.10.1.3b or designated as blocked in

Table 9.10.1.4b, the horizontal joints shall occur over blocking at least equal in size to the studs.

9.10.1.1 Single Diagonally Sheathed Shear Panels: Single diagonally sheathed shear panels shall consist of 1x sheathing boards laid at an angle of approximately 45 degrees to supports. Common nails at each intermediate support shall be two 8d for 1 by 6 and three 8d for 1 by 8 boards. One additional nail shall be provided in each board at shear panel boundaries. For box nails, one additional nail shall be provided in each board at each intermediate support and two additional nails shall be provided in each board at shear panel boundaries. End joints in adjacent boards shall be separated by at least one framing space between supports. Single diagonally sheathed shear panels may consist of 2x sheathing boards where 16d nails are substituted for 8d nails, end joints are located as above, and the support is not less than 3 inches wide or 4 inches deep.

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

9.10.1.2 Double Diagonally Sheathed Shear Panels: Double diagonally sheathed shear panels shall conform to the requirements for single diagonally sheathed diaphragms and the requirements of this section.

Double diagonally sheathed shear panels shall be sheathed with two layers of diagonal boards placed perpendicular to each other on the same face of the supports. Each chord shall be designed for the axial force induced flexure between supports due to a uniform load equal to 50 percent of the load for which the diaphragm was designed.

The allowable working stress shear for these panels is 600 pounds per lineal foot.

9.10.1.3 Structural Use Shear Panels: Horizontal and vertical shear panels sheathed with structural use sheets may be used to resist earthquake forces based on the allowable working stress shear set forth in Table 9.10.1.3a for horizontal diaphragms and Table 9.10.1.3b for shear walls or may be calculated by principles of mechanics without limitation by using values of nail strength and structural use panel sheathing shear strength given in the reference standards. At boundaries and changes in direction of framing, the sheathing shall be arranged so that no sheet has a minimum dimension of less than 2 feet. The edges of all structural use panel sheets shall be supported by framing or blocking having the minimum width given in Tables 9.10.1.3a and 9.10.1.3b for blocked diaphragms and shear walls. The size and spacing of fasteners at structural use sheathing panel boundaries, structural use sheathing sheet edges, and intermediate supports shall be as given in Tables 9.10.1.3a and 9.10.1.3b.

9.10.1.4 Shear Panels Sheathed with Other Materials: Light framed walls sheathed with lath and plaster, gypsum sheathing boards, gypsum wallboard, or fiberboard sheets may be used to resist earthquake forces in framed buildings except as limited by Sec. 9.6.1 and 9.7.1. The allowable working stress shears are given in Tables 9.10.1.4a and 9.10.1.4b. The maximum h/w ratio shall be 1.5:1 where h = the height of a shear panel or shear wall and w = the width of a shear panel, shear wall, or diaphragm.

The shear values for these shear panels shall not be cumulative with the shear values for other materials applied to the same wall line. The shear values for the same material applied to both faces of the same wall are cumulative.

TABLE 9.10.1.3a
Allowable Working Stress Shear for Seismic Forces in Pounds per Foot on Horizontal Structural Use Panel Diaphragms with Framing Members of Douglas Fir-Larch or Southern Pine[a,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							Unblocked Diaphragms[e]	
						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]							Fastener Spacing at 6 Inches at Supported Edges	
						6	4		2-1/2[d]		2[d]		Case 1	Cases 2, 3, 4, 5, and 6
						Spacing (in.) per Line at Other Panel Edges								
						6	6	4	4	3	3	2		
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	1,230				
				4	2		755	980	1,080	1,410				
				4	3		940	1,305	1,375	1,810				
	14-gauge staples	2	23/32	3	2		600	600	840	900	1,040	1,200		
				4	3		840	900	1,140	1,350	1,440	1,800		
C-D, C-C and Other Similar Grades	6d common	1-1/4	5/16	2	1	170	225		335		380		150	110
				3	1	190	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

	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	1,225				
				4	2		750	980	1,075	1,395				
				4	3		935	1,305	1,390	1,510				
	14-gauge staples	2	23/32	3	2		600	600	820	900	1,020	1,200		
				4	3		820	900	1,120	1,350	1,400	1,510		

[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 12 inch centers 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 1,200 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.

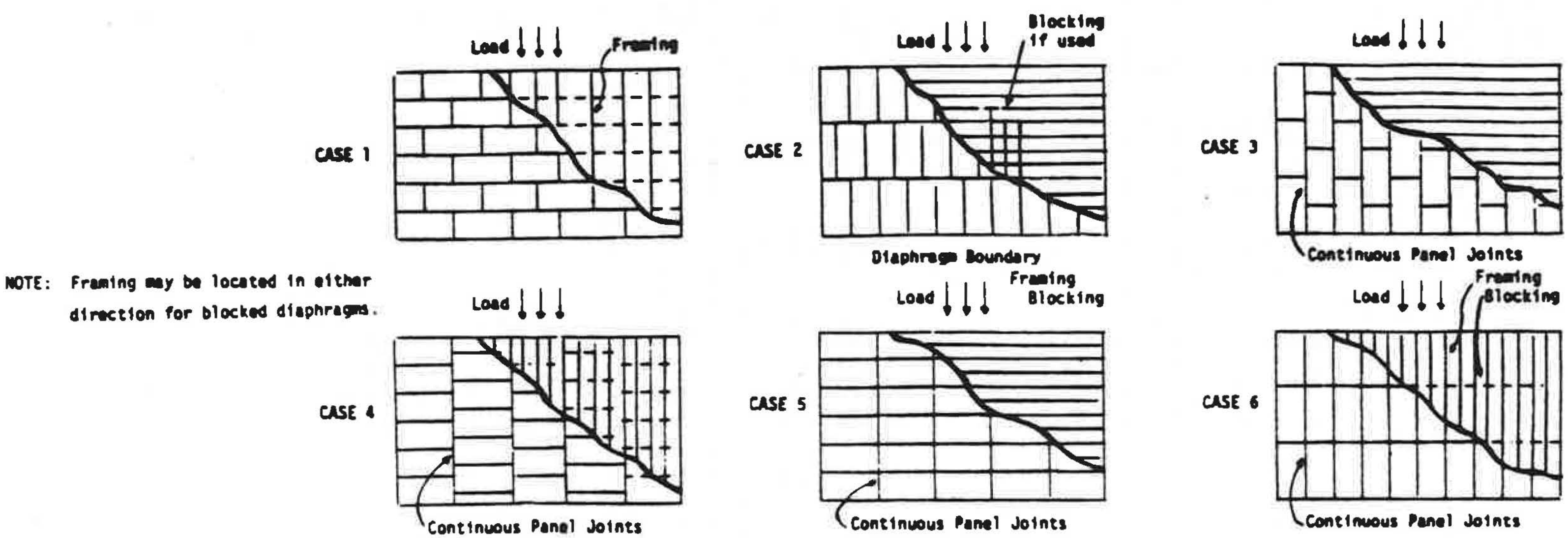

TABLE 9.10.1.3b
Allowable Working Stress Shear for Seismic Forces in Pounds per Foot on Structural Use Panel Shear Walls with Framing of Douglas Fir-Larch or Southern Pine[a]

Panel Grade	Nail Size	Penetration in Framing (in.)	Panel Thickness (in.)	Panel Applied Direct to Framing[b]				Nail Size	Panel Applied Over 1/2-Inch Gypsum Sheathing			
				6	4	3	2		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 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	15/32	340	510	665[c]	870					
Panel Siding in Grades Covered in PS 1-83	6d[f]	1-1/4	5/16	140	210	275	360	8d	140	210	275	360
	6d[f]	1-1/2		160[e]	240[e]	310[e]	410[e]	10d	160	240	310[c]	410

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

[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 panels applied directly to framing may be increased by 20 percent provided studs are spaced a maximum of 16 inches on center or panel is applied with face grain across studs or if the panel thickness is increased 1/2 inch or greater.

[f]Galvanized casing nails.

TABLE 9.10.1.4a
Allowable Working Stress Shears for Seismic Forces in Pounds per Foot on Vertical Shear Panels of Fiberboard Sheathing Board[a]

Size/Application	Nail Size	Shear Value for 3-Inch Nail Spacing at Sheet Perimeter and 6-Inch Spacing at Intermediate Supports
1/2 in. by 4 ft by 8 ft	No. 11 ga. galvanized 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. galvanized 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. In Category C buildings, the allowable values in the table shall be reduced 50 percent.

[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 9.10.1.4b
Allowable Working Stress Shears for Seismic Forces in Pounds per Foot on 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	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. In Category C and D buildings, the allowable values in the table shall be reduced 50 percent.

[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 *Load and Resistance Factor Design Specification for Structural Steel Buildings (LRFD)*, American Institute of Steel Construction (AISC), September 1, 1986, including Supplement No. 1 effective January 1, 1989

Ref. 10.2 *Allowable Stress Design and Plastic Design Specification for Structural Steel Buildings (ASD)*, American Institute of Steel Construction, June 1, 1989

Ref. 10.3 Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute (AISI), August 10, 1986 Edition with December 11, 1989, Addendum

Ref. 10.4 Specification for the Design of Cold-formed Stainless Steel Structural Members, ANSI/ASCE 8-90, American Society of Civil Engineers

Ref. 10.5 Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders, Steel Joist Institute, 1990 Edition

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

10.1.1 DEFINITIONS:

Concentrically Braced Frame (CBF): A braced frame in which the members are subjected primarily to axial forces.

K-Braced Frame: A concentrically braced frame in which a single pair of diagonal braces located on one side of a column is connected to a single point within the clear column height.

V-Braced Frame: A concentrically braced frame (CBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system also is referred to as an "inverted V-braced frame."

X-Braced Frame: A concentrically braced frame (CBF) in which a pair of diagonal braces crosses near midlength of the braces.

Continuity Plate: Column stiffeners at the top and bottom of the panel zone.

Design Strength: Resistance (force, moment, stress, as appropriate) provided by the element or connection; the product of the nominal strength and the resistance factor.

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: A member of an EBF placed diagonally in the bay of the frame.

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

Link Beam: The horizontal beam in an EBF. 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: 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: Vertical web stiffeners placed within the link beam.

Link Beam Rotation Angle: 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.

Link Beam Shear Yield Strength: The strength prescribed in Sec. 10.12.

Nominal Load: The magnitude of the loads Q_D, Q_E, Q_L, or Q_S as appropriate.

Nominal Strength: The capacity of a structure or element to resist the effects of loads as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Required Strength: Load effect (force, moment, stress, as appropriate) acting on elements or connections determined by structural analysis from the factored loads (using the most critical load combinations).

Resistance Factor: A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure.

10.1.2 SYMBOLS:

A_e = Effective net area, in.^{2}

A_f = Flange area of member, in.^{2}

A_g = Gross area, in.^{2}

A_{st} = Area of link stiffener, in.^{2}

A_w = Effective area of weld, in.^{2}

A_w = Link web area, in.^{2}

F_w = Nominal strength of weld electrode material, ksi

F_{BM} = Nominal strength of the base material to be welded, ksi

F_{EXX} = Classification strength of weld metal, ksi

F_y = Specified minimum yield stress of the type of steel being used, ksi

F_{yb} = F_y of a beam, ksi

F_{yc} = F_y of a column, ksi

F_{yf} = F_y of a flange, ksi

H = Average story height above and below a beam-to-column connection, inch

L = Length of bracing member, inch

M_n = Nominal flexural strength of a member or joint, kip-inch

M_p = Plastic bending moment, kip-inch

M_{pa} = Plastic bending moment modified by axial load ratio, kip-inch

M_u = Required flexural strength of a member or joint, kip-inch

M_{uy} = M_u about the Y-axis, kip-inch

P_u = Required axial strength of a column or a link, kips

P_n	=	Nominal axial strength of a column, kips
P_u^*	=	Required axial strength of a brace, kips
P_{uc}	=	Required axial strength of a column based on Eq. 3-1 and 3-2, kips
P_y	=	Nominal yield axial strength of a member = $F_y A_g$, kips
R	=	Response modification coefficient given in Table 3.3.
R_n	=	Nominal strength of a member or joint.
V_n	=	Nominal shear strength of a member, kips
V_u	=	Required shear strength of a member, kips
V_y	=	Nominal shear strength of an active link, kips
V_{ya}	=	Nominal shear strength of a member modified by the axial load magnitude, kips
Z	=	Plastic section modulus, in.^{3}
Z_b	=	Plastic section modulus of a beam, in.^{3}
Z_c	=	Plastic section modulus of a column, in.^{3}
b	=	Width of compression element, inches
b	=	Width of EBF link stiffener, inches
b_f	=	Flange width, inches
b_{cf}	=	Column flange width, inches
d	=	Overall depth of member, inches
d_b	=	Overall beam depth, inches
d_c	=	Overall column depth, inches
d_z	=	Overall panel zone depth between continuity plates, inches
e	=	EBF link length, inches
h_c	=	Assumed web depth for stability, inches

k_v	=	Shear buckling coefficient
r	=	Governing radius of gyration, inches
r_y	=	Radius of gyration about y axis, inches
t	=	Thickness of link stiffener, inches
t_{bf}	=	Thickness of beam flange, inches
t_{cf}	=	Thickness of column flange, inches
t_f	=	Thickness of flange, inches
t_p	=	Thickness of panel zone including doubler plates, inches
t_w	=	Thickness of web, inches
t_z	=	Thickness of panel zone (doubler plates not necessarily included), inches
w_z	=	Width of panel zone between column flanges, inches
α	=	Fraction of member force transferred across a particular net section
ρ	=	Ratio of required axial force (P_u) to nominal shear strength (V_y) of a link
λ	=	Slenderness parameter
λ_p	=	Limiting slenderness parameter for compact element
λ_r	=	Limiting slenderness parameter for noncompact element
ϕ	=	Resistance factor
ϕ_b	=	Resistance factor for beams
ϕ_c	=	Resistance factor for columns in compression
ϕ_t	=	Resistance factor for columns in tension
ϕ_w	=	Resistance factor for welds
Ω	=	Factor of safety

10.2 STRENGTH OF MEMBERS AND CONNECTIONS: The strength of members and connections subjected to seismic forces in combination with other prescribed loads shall be determined as required by Ref. 10.1 (LRFD) and 10.4 (LRFD) except that the load factors shall comply with the requirements of these provisions. An alternative procedure for determining the design strengths of structural steel designed in accordance with Ref. 10.2 (ASD) is presented in Sec. 10.14.

The design strengths of other types of steel in Ref. 10.3, 10.5, and 10.6 shall be determined by converting allowable stresses into nominal strengths as required by this section and multiplying such nominal strengths by the resistance factors herein.

In the absence of resistance factors (ϕ) in Ref. 10.3, the value of ϕ shall be as follows:

Shear strength with $h/t > \sqrt{Ek_v/F_y}$	$\phi = 0.9$
Shear strength with $h/t \leq \sqrt{Ek_v/F_y}$	$\phi = 1.0$
Web crippling of members with single unreinforced webs	$\phi = 0.75$
Web crippling of "I" sections	$\phi = 0.8$
All other cases where Ω is the factor of safety	$\phi = 1.55/\Omega$

In the absence of resistance factors (ϕ) in Ref. 10.4, 10.5, and 10.6, 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	$\phi = 0.67$
Metal deck diaphragms	$\phi = 0.60$
Partial penetration welds in columns when subjected to tension stresses	$\phi = 0.80$

10.2.1 COLD-FORMED STEEL: Reference 10.3 shall be modified as follows:

10.2.1.1 Member Nominal Strength: In Ref. 10.3, the nominal strength of members and joints shall be as specified except that the nominal strength for shear and web crippling shall be computed using 1.7 times the allowable stress specified.

10.2.1.2 Steel Deck Diaphragms: Nominal strength values for steel deck diaphragms made from materials conforming to the requirements of Ref. 10.3 or 10.4 shall be the strength values as approved by the regulatory agency.

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

10.2.2 STEEL JOISTS: The nominal strength of members and joints shall be computed using 1.7 times the allowable loads and allowable stresses as specified in Ref. 10.5.

10.2.3 STEEL CABLES: Reference 10.6, Sec. 5d, shall be modified by substituting $1.5(T_4)$ when T_4 is the net tension in cable due to dead load, prestress, live load, and seismic load. A load 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 of the reference documents and to the additional requirements of this section. Ordinary moment frames, space frames in building frame systems, and space frames incorporated into bearing wall systems shall be designed and constructed in accordance with Ref. 10.1, 10.2, 10.3, or 10.4.

Unless otherwise noted, the additional requirements of the following subsections shall apply only to buildings in Seismic Hazard Exposure Group III where the effective peak velocity-related acceleration (A_v) value is greater than or equal to 0.10.

10.4.1 MATERIAL SPECIFICATIONS: Steel used in seismic force resisting systems shall be limited by the provisions of Sec. 10.7.

10.4.2 COLUMN REQUIREMENTS: Columns in seismic force resisting systems shall be designed in accordance with the provisions of Sec. 10.8.

10.4.3 ORDINARY MOMENT FRAME REQUIREMENTS: Ordinary moment frames shall be designed in accordance with the provisions of Sec. 10.9.

10.4.4 SPECIAL MOMENT FRAMES: Special moment frames need conform only to the requirements of Sec. 10.10.2, 10.10.7, and 10.10.8.

10.4.5 BRACED FRAMES: In Seismic Hazard Exposure Group III buildings where the effective peak velocity-related acceleration (A_v) value is greater than or equal to 0.10., braced frame systems shall conform to the requirements of Sec. 10.11 or 10.12 when used alone or in combination with moment frames as the seismic force resisting system. In all other Seismic Performance Category C buildings in which braced frames are used as the seismic force resisting system, brace connections shall be designed to develop the tensile yield capacity of the brace or to provide tensile deformation equivalent to the deflection amplification factor (C_d) as specified in Table 3.3 times the brace deformation caused by the seismic design forces.

10.4.5.1 K-Bracing: Except as permitted by Sec. 10.11.5, K-bracing shall meet the requirements in Sec. 10.11.4.1.

10.5 SEISMIC PERFORMANCE CATEGORY D: Buildings assigned to Category D shall conform to the requirements of the reference documents and to the additional requirements of this section.

10.5.1 MATERIAL SPECIFICATIONS: Steel used in seismic force resisting systems shall be limited by the provisions of Sec. 10.7.

10.5.2 COLUMN REQUIREMENTS: Columns in seismic force resisting systems shall be designed in accordance with the provisions of Sec. 10.8.

10.5.3 ORDINARY MOMENT FRAMES: Ordinary moment frames shall be designed in accordance with the provisions of Sec. 10.9.

10.5.4 SPECIAL MOMENT FRAMES: Special moment frames shall be designed in accordance with the provisions of Sec. 10.10.

10.5.5 BRACED FRAMES: Where braced frame systems are used alone or in combination with moment frames as the seismic force resisting system, the braced frames shall conform to the requirements of Sec. 10.11 or 10.12.

10.5.5.1 K-Bracing: The use of K-bracing shall not be permitted as part of the seismic force resisting system except for those framing systems meeting the requirements of Sec. 10.11.5.

10.5.6 LIGHT FRAMED WALLS: Cold-formed steel light framed walls shall be designed and constructed in accordance with the provisions of Sec. 10.13. Steel used in such systems shall be as specified in Ref. 10.3 and shall not be limited by the provisions of Sec. 10.7.

10.6 SEISMIC PERFORMANCE CATEGORY E: Buildings assigned to Category E shall conform to all of the requirements for Category D (Sec. 10.5) and to the additional requirements of this section.

10.6.1 BRACED FRAMES: For buildings greater than one story in height, concentrically braced frame systems shall be used only as part of a dual system. Where a concentrically braced frame system is used in combination with ordinary moment frames or special moment frames as the seismic force resisting system, the concentrically braced frames shall conform to the requirements of Sec. 10.11. Where an eccentrically braced frame (EBF) system is used as part of the seismic force resisting system, it shall conform to the requirements of Sec. 10.12.

10.7 MATERIAL SPECIFICATIONS: The steel used in seismic force resisting systems in buildings more than one story in height shall be limited to the following ASTM Specifications: A36, A441, A500 (Grades B and C), A501, A572 (Grades 42 and 50), and A588. The use of A283 (Grade D) is permitted for base plates.

10.8 COLUMN REQUIREMENTS:

10.8.1 NOMINAL COLUMN STRENGTH: When $P_u/\phi P_n > 0.5$, column axial loads (without combination with concurrent moments) shall be limited by the following requirements:

10.8.1.1: Axial compression loads resulting from the application of Eq. 3-1a shall be less than or equal to ϕP_n.

10.8.1.2: Axial tension load resulting from the application of Eq. 3-2a shall be less than or equal to ϕP_n.

10.8.1.3: The axial load combination Eq. 3-1a and 3-2a need not exceed the lesser of the following:

a. The maximum that can be transferred to the column considering 1.25 times the design strengths of the connecting beam or brace elements of the structure.

b. The limit as determined by the foundation capacity to resist overturning uplift.

10.8.2 COLUMN SPLICES: In addition to the load combinations, Eq. 3-1 and 3-2, column splices shall have sufficient strength to develop the column axial loads given by load combination Eq. 3-2a.

10.8.2.1: In column splices using either complete or partial penetration welded joints, changes in thickness and width of flanges and webs are permitted without providing beveled transitions.

10.8.2.2: Splices using partial penetration welded joints shall not be within 3 feet of the beam-to-column connection. Column splices that are subject to net tension forces shall comply with the more critical of the following where the resistance factor (ϕ) is equal to 0.8, $F_w = 0.60F_{EXX}$, F_{yc} = the yield strength of the column material, and A_f = the flange area of the smaller column connected:

a. The design strength of partial penetration welded joints, the lesser of $\phi_w F_w A_w$ or $\phi F_{bm} A_w$, shall be at least 150 percent of the required strength.

b. The design strength of welds shall not be less than 0.5 $F_{yc}A_f$.

where $F_w = 0.60F_{EXX}$, F_{yc} = the yield strength of the column material, and A_f = the flange area of the smaller column connected.

10.9 ORDINARY MOMENT FRAME (OMF) REQUIREMENTS:

10.9.1 DESIGN STRENGTH: Ordinary moment frames shall have the design strength to resist the factored load combinations in Eq. 3-1 and 3-2.

10.9.2 JOINT REQUIREMENTS: All beam-to-column connections in OMFs that resist earthquake forces shall meet one of the following requirements:

1. Sec. 10.10.2.

2. The design shall be capable of inelastic deformations and the required strengths of the connections shall meet the requirements of Sec. 10.9.1 using the load combinations in Eq. 3-1a and 3-2a.

3. Either fully restrained (FR) or partially restrained (PR) connections are permitted provided:

   a. The design strengths of the member and connections meet the requirements of Sec. 10.9.1,

   b. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at an elastic story drift calculated at horizontal load V_x times $(2R/5)$, which shall be not less than V_x, and

   c. The additional drift due to PR connections shall be considered in design.

10.10 SPECIAL MOMENT FRAME (SMF) REQUIREMENTS:

10.10.1 SCOPE: Special moment frames shall be used when required by other sections of these provisions.

10.10.2 BEAM-TO-COLUMN JOINTS:

10.10.2.1: The required flexural strength (M_u) of each beam-to-column joint shall be the lesser of the following quantities:

1. The plastic bending moment (M_p) of the beam

2. The moment resulting from the panel zone nominal shear strength (V_n) as determined using Eq. 10-1

The joint need not develop either of the strengths defined above if it can be shown that under frame deformation amplified by $(2R/5)$, but not less than 1.0, of that produced by the load combinations of Eq. 3-1 and 3-2, the design strength of the members at the connection are adequate to support the vertical loads, and the required lateral resistance is provided by other means.

10.10.2.2: The required shear strength (V_u) of beam-to-column joints shall be determined using the load combination 1.2D + (0.5L or 0.2S) plus the shear resulting from M_u, as defined in Sec. 10.10.2.1, on at least one end of the beam. The required shear strength need not exceed, however, the shear resulting from the load combination Eq. 3-1a.

10.10.2.3: The design strength (ϕR_n) of a beam-to-column joint can be considered to be adequate to develop the required flexural strength (M_u), of the beam if it conforms to the following:

10.10.2.3.1: The beam flanges are welded to the column using complete penetration welded joints.

10.10.2.3.2: The beam web joint shall have a design shear strength (ϕV_n) greater than the required shear strength (V_u) and conform to either:

a. Where the nominal flexural strength of the beam (M_n) considering only the flanges is greater than 70 percent of the nominal flexural strength of the entire beam section [i.e., $b_f t_{bf}(d - t_{bf})F_{yf} \geq 0.7M_p$]; the web joint is permitted to be made by means of welding or slip-critical high-strength bolting, or

b. Where $b_f t_{bf}$(d - $t_{bf})F_{yb}$ < $0.7M_p$, the required beam shear strength shall be developed by a minimum 20 percent welding and the remainder shall be developed by further welding or by slip-critical high-strength bolting.

10.10.2.4: Joint configurations utilizing welds or high-strength bolts but not conforming to Sec. 10.10.2.3 are permitted to be used if shown by test or calculations to meet the criterion therein. Where conformance is shown by calculation, the design strength of the joint shall be 125 percent of the design strengths of the connecting elements.

10.10.3 PANEL ZONE OF BEAM-TO-COLUMN CONNECTION (Beam Web Parallel to Column Web):

10.10.3.1 Shear Strength: The required shear strength (V_u) of the panel zone shall be based on beam bending moments determined from Eq. 3-1 and 3-2. V_u, however, need not exceed that determined from $0.9\Sigma\phi_b M_p$ of the beams framing into the column flanges at the connection. The design shear strength ($\phi_v V_n$) of the panel zone shall be determined by the following equation:

$$\phi_v V_n = 0.55\phi_v F_y d_c t_p \left(1 + \frac{3 b_{cf} t_{cf}^3}{d_b d_c t_p}\right) \qquad (10\text{-}1)$$

where, for this case, ϕ_v = 0.8 and

F_y = specified minimum yield stress of the steel (ksi),

d_c = overall column section depth (inches),

t_p = total thickness of the panel zone including doubler plates (inches),

b_{cf} = width of the column flange (inches),

t_{cf} = thickness of the column flange (inches), and

d_b = overall beam depth (inches).

10.10.3.2 Panel Zone Thickness: The panel zone thickness (t_z), shall conform to the following:

$$t_z \geq \frac{d_z + w_z}{90} \tag{10-2}$$

where

d_z = the panel zone depth between continuity plates (inches) and

w_z = the panel zone width between column flanges (inches).

For this purpose t_z shall not include any doubler plate thickness unless the doubler plate is connected to the web with plug welds adequate to prevent buckling of the plate.

10.10.3.3 Panel Zone Doubler Plates: Doubler plates provided to increase the design strength of the panel zone or to reduce the web depth thickness ratio shall be placed close to the column web and welded across the plate width top and bottom with a minimum fillet weld per Table J2.5 of Ref. 10.1. The doubler plates shall be fastened to the column flanges using either butt or fillet welded joints to develop the design shear strength of the doubler plate.

10.10.4 BEAM LIMITATIONS:

10.10.4.1 Beam Flange Area: Abrupt changes in beam flange areas are not permitted within potential plastic hinge regions.

10.10.4.2 Width-Thickness Ratios: The width-thickness ratios of compression elements of beams shall comply with the limiting slenderness parameter for compact elements (λ_p) as given in Table 10.10.4.2 in lieu of those in Table B5.1 of Ref. 10.1.

TABLE 10.10.4.2
Limiting Width Thickness Ratios (λ_p) for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_p
Flanges of I-shaped non-hybrid sections and channels in flexure Flanges of I-shaped hybrid beams in flexure	b/t	$\dfrac{52}{\sqrt{F_y}}$
Webs in combined flexural and axial compression	h_c/t_w	For $P_u/\phi_b P_y \le 0.125$ $\dfrac{520}{\sqrt{F_y}}\left(1 - \dfrac{1.54P_u}{\phi_b P_y}\right)$ For $P_u/\phi_b P_y > 0.125$ $\dfrac{191}{\sqrt{F_y}}\left(2.33 - \dfrac{P_u}{\phi_b P_y}\right) \ge \dfrac{253}{\sqrt{F_y}}$

10.10.5 CONTINUITY PLATES: Continuity plates shall be provided if the column flange local bending nominal strength (R_n) is less than $1.8F_{yb}b_f t_{bf}$ where:

R_n = $6.25(t_{cf})^2 F_{yf}$,

F_{yb} = specified minimum yield stress of beam (ksi),

F_{yf} = specified minimum yield stress of column flange (ksi),

b_f = beam flange width (inches),

t_{bf} = beam flange thickness (inches), and

t_{cf} = column flange thickness (inches).

Continuity plates shall be fastened by welds to both flanges and webs or doubler plates of columns.

10.10.6 COLUMN-BEAM MOMENT RATIO: At any beam to column connection, one of the following must be satisfied:

$$\frac{\Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right)}{\Sigma Z_b F_{yb}} \geq 1.0 \tag{10-3}$$

or

$$\frac{\Sigma Z_c \left(F_{yc} - \frac{P_{uc}}{A_g} \right)}{V_n d_b \left(\frac{H}{H - d_b} \right)} \geq 1.0 \tag{10-4}$$

where P_{uc} (in compression) ≥ 0 and

Z_c = the plastic section of modulus of the column (in.^{3}),

F_{yc} = the specified minimum yield stress of the column (ksi),

P_{uc} = the required axial strength of the column based on Eq. 3-1 and 3-2 (kips),

A_g = the gross area (in.^{2}),

Z_b = the plastic section modulus of the beam (in.^{3}),

F_{yb} = the specified minimum yield stress of the beam (ksi),

V_n = the nominal strength of the panel zone as determined from Eq. 10-1,

d_b = the average overall depth of the beams framing into the connection, and

H = the average of the story height above and below.

These requirements need not apply in the following cases provided that the columns conform to the requirements of Sec. 10.10.4.2:

a. For columns with $P_{uc} < 0.3F_yA_g$

b. For columns in any story that has a total design lateral shear strength 50 percent greater than that of the story shear above

c. For any column not included in the design to resist the required seismic shears, although the column included in the design to resist axial overturning forces.

10.10.7 BEAM-TO-COLUMN CONNECTION RESTRAINT:

10.10.7.1 Restrained Connection:

10.10.7.1.1: Column flanges at a beam-to-column connection require lateral support only at the level of the top flanges of the beams when a column can be shown to remain elastic outside of the panel zone. This can be satisfied by one of the following conditions:

a. The ratio in either Eq. 10-3 or 10-4 is greater than 1.25.

b. The column remains elastic when loaded with load combination Eq. 3-1a.

10.10.7.1.2: When a column cannot be shown to remain elastic outside of the panel zone, the following provisions apply:

a. The column flanges shall be laterally supported at the levels of both top and bottom beam flanges.

b. Each column flange lateral support shall be designed for a required strength equal to 1.5 percent of the nominal beam flange strength ($F_{yb}b_f t_{bf}$).

c. Column flanges shall be laterally supported either directly or indirectly through the column web or beam flanges.

10.10.7.2 Unrestrained Connections: A column containing a beam-to-column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height and conform to Sec. H1 of Ref. 10.1 except for the following:

10.10.7.2.1: The required column weak axis strength shall be determined from Eq. 3-1 where Q_E is the least of:

a. The earthquake force amplified by ($2R/5$) greater than or equal to 1.0, or

b. 125 percent of the frame design strength based on either beam or panel zone design strengths.

10.10.7.2.2: The nominal column axial strength (P_n) shall be based on a pin ended column.

10.10.7.2.3: The L/r for these columns shall not exceed 60.

10.10.7.2.4: The required column weak axis moment (M_{uy}) shall include that caused by the beam flange force specified in Sec. 10.10.7.1.2.b plus the added P-delta moment due to the resulting column flange displacement.

10.10.8 LATERAL SUPPORT OF BEAMS: Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $2{,}500\, r_y/F_y$. In addition, lateral supports shall be placed at concentrated loads where a hinge may form.

10.11 CONCENTRICALLY BRACED FRAME (CBF) REQUIREMENTS:

10.11.1 SCOPE: The provisions of this section apply to all braced frames except eccentrically braced frames designed in accordance with Sec. 10.12. Those members that resist seismic forces totally or partially by shear or flexure shall be designed in accordance with Sec. 10.10.

10.11.2 BRACING MEMBERS:

10.11.2.1 Compressive Design Strength: The design strength of a bracing member in axial compression shall be determined by $0.8\phi_c P_n$ where ϕ_c is the resistance factor for columns in compression and P_n is the nominal axial strength of a column (kips).

10.11.2.2 Width-Thickness Ratios: Braces shall be compact or noncompact but not slender members (i.e., $\lambda < \lambda_r$). Circular sections shall have an outside diameter to wall thickness ratio not exceeding $1{,}300/F_y$. Rectangular tubes shall have a flat width to wall thickness not exceeding $110/\sqrt{F_y}$ unless the tube walls are stiffened.

10.11.2.3 Built-Up Member Stitches:

10.11.2.3.1: For a brace in which stitches can be subjected to post buckling shear, the spacing of stitches shall be such that the slenderness ratio (L/r, of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member. The total shear strength of the stitches shall be at least equal to the nominal tensile strength of each element. The spacing of the stitches shall be uniform and not less than two stitches shall be used.

10.11.2.3.2: For braces that can buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio (L/r, of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

10.11.2.4 Slenderness: Bracing members in Seismic Performance Category D and E buildings shall have an $L/r \leq 720/\sqrt{F_y}$ except as permitted in Sec. 10.11.5.

10.11.2.5 Lateral Force Distribution: In Seismic Performance Category D and E buildings, along any line of bracing, braces shall be arranged in opposing directions so that in either direction of loading along the line at least 30 percent of the seismic force distributed to that line is resisted by braces in tension. Where the nominal strength (P_n) of all braces in compression is larger than the required strength (P_u) resulting from the load combination Eq. 3-1a or Eq. 3-2a, such opposing bracing is not required. A line of bracing, for the purpose of this provision, is defined as a single line or parallel lines within 10 percent of the building dimension perpendicular to the line of bracing.

10.11.3 BRACING CONNECTIONS:

10.11.3.1 Forces: The required strength of bracing joints (including beam-to-column joints if part of the bracing system) shall be the least of the following:

1. The design axial tension strength of the bracing member.

2. The axial force and end moments and shear force in the brace resulting from load combination Eq. 3-1a or Eq. 3-2a.

3. The maximum force that can be transferred to the brace by the system.

10.11.3.2 Net Area: In bolted brace joints, the minimum ratio of effective net section area to gross section area shall be limited by:

$$\frac{A_e}{A_g} \geq \frac{1.2\,\alpha\,P_u^*}{\phi_t P_n} \tag{10-5}$$

where:

A_e = effective net area,

A_g = gross area,

α = fraction of the member force from Sec. 10.11.3.1 that is transferred across a particular net section

P_u^* = required strength of the brace as determined in Sec. 10.11.3.1,

ϕ_t = tension strength resistance factor = 0.75, and

P_n = nominal tension strength.

10.11.3.3 Gusset Plates:

10.11.3.3.1: For braces that can buckle in the plane of the gusset plate, the gusset and other parts of the connection shall have a design strength equal to or greater than the nominal in-plane bending strength of the brace.

10.11.3.3.2: For braces that can buckle out-of-plane, the brace shall terminate on the gusset a minimum of two times the gusset thickness from a line about which the gusset plate can bend unrestrained by the column or beam joints. The gusset plate shall be designed to carry the compressive design strength of the brace member without local buckling of the gusset plate. For braces designed for axial load only, the bolts or welds shall be designed to transmit the brace forces along the centroids of the brace elements.

10.11.4 SPECIAL V-BRACING AND K-BRACING CONFIGURATION REQUIREMENTS:

10.11.4.1: The design strength of K-brace members where permitted and V-brace members shall be at least 1.5 times the required strength using Eq. 3-1 and 3-2.

10.11.4.2: A beam intersected by V-braces shall be continuous between columns.

10.11.4.3: A beam intersected by V-braces shall be capable of supporting all tributary dead and live loads assuming the bracing is not present.

10.11.4.4: The top and bottom flanges of the beam at the point of intersection of V-braces shall be designed to support a lateral force equal to 1.5 percent of the nominal beam flange strength $(F_{yb}b_f t_{bf})$.

10.11.5 LOW BUILDINGS: Braced frames not meeting the requirements of Sec. 10.11.2 through 10.11.4 are permitted in buildings of two stories or less in height and in the top story if load combination Eq. 3-1a and Eq. 3-2a are used for determining the required strength of the members and connections.

10.12 ECCENTRICALLY BRACED FRAME REQUIREMENTS:

10.12.1 SCOPE: Eccentrically braced frames shall be designed so that under earthquake loading, yielding will occur primarily in the links. The diagonal braces, the columns, and the beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links. In EBFs, plastic hinges shall not be permitted to develop in columns at floor beam levels up to an amplified frame displacement of $(2R/5)$ greater than or equal to 1.0 times that produced by code seismic design forces.

10.12.2 LINKS:

10.12.2.1: Links shall comply with the width-thickness ratios in Table 10.10.4.2.

10.12.2.2: The specified minimum yield stress of steel (F_y) used for links shall not exceed 50 ksi.

10.12.2.3: The web of a link shall be single thickness without doubler plate reinforcement and without openings.

10.12.2.4: The required shear strength of the link (V_u) shall not exceed the design shear strength of the link (ϕV_n) defined as the lesser of:

$$\phi V_y \tag{10-6}$$

or

$$\frac{2\phi M_p}{e} \tag{10-7}$$

where:

V_y = $0.6F_y d t_w$,

ϕ = 0.9, and

e = link length; except as limited by Sec. 10.12.2.6.

10.12.2.5: If the required axial strength (P_u) in a link is equal to or less than $0.15P_y$, where P_y equals the gross area (A_g) times the specified minimum yield stress (F_y), the effect of axial force on the link design shear strength need not be considered.

10.12.2.6: If the required axial strength (P_u) in a link exceeds 0.15 times the nominal yield axial strength (P_y), the following additional limitations shall apply:

10.12.2.6.1: The link design shear strength shall be the lesser of:

$$\phi V_{ya} \tag{10-8}$$

or

$$\frac{2\phi M_{pa}}{e} \tag{10-9}$$

where:

V_{ya} = $V_y \sqrt{1 - (P_u/P_y)^2}$,

M_{pa} = $1.18\, M_p\, [1 - (P_u/P_y)]$, and

ϕ = 0.9.

10.12.2.6.2: The length of the link shall not exceed:

$$\left[1.15 - 0.5\rho\left(\frac{A_w}{A_g}\right)\right]\left[\frac{1.6M_p}{V_y}\right] \tag{10-10}$$

for $(A_w/A_g) \geq 0.3$ and

$$\frac{1.6M_p}{V_y} \tag{10-11}$$

for $(A_w/A_g) < 0.3$ where $A_w = dt_w$ and $\rho = P_u/V_y$.

10.12.2.7: The link rotation angle is the plastic angle between the link and the beam outside of the link at the design story drift as determined by Sec. 4.6.1. Except as noted in Sec. 10.12.4.3, the link rotation angle shall not exceed the following values:

a. 0.09 radians for links of length 1.6 M_p/V_y or less, provided the fundamental period of the EBF is equal to or greater than 1.0 second; otherwise the link rotation angle shall not exceed 0.08 radians.

b. 0.02 radians for links of length 2.6 M_p/V_y or greater.

c. Linear interpolation shall be used for links of length between $1.6M_p/V_y$ and 2.6 M_p/V_y.

10.12.3 LINK STIFFENERS:

10.12.3.1: Full depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ nor 3/8 inches, whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

10.12.3.2: Links shall be provided with intermediate web stiffeners as follows:

10.12.3.2.1: Links of lengths $1.6M_p/V_y$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.09 radians or $(52t_w - d/5)$ for link rotation angles or 0.03 radians or less. Linear interpolation shall be used for values between 0.03 and 0.09 radians.

10.12.3.2.2: Links of lengths greater than 2.6 M_p/V_y and less than 5 M_p/V_y shall be provided with intermediate web stiffeners placed at a distance of $1.5b_f$ from each end of the link.

10.12.3.2.3: Links of length between 1.6 M_p/V_y and 2.6 M_p/V_y shall be provided with intermediate web stiffeners meeting the requirements of 1 and 2 above.

10.12.3.2.4: No intermediate web stiffeners are required in links of lengths greater than $5M_p/V_y$.

10.12.3.2.5: Intermediate link web stiffeners shall be full depth. For links less than 25 inches in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 3/8 inch, whichever is larger, and the width shall be not less than $[(b_f/2) - t_w]$. For links 25 inches in depth or greater, similar intermediate stiffeners are required on both sides of the web.

10.12.3.3: Fillet welds connecting link stiffener to the link web shall have a design strength adequate to resist a force of $A_{st}F_y$, in which A_{st} is the area of the stiffener and F_y is the specified minimum yield stress. The design strength of fillet welds fastening the stiffener to the flanges shall be adequate to resist a force of $A_{st}F_y/4$.

10.12.4 LINK-TO-COLUMN CONNECTIONS: Where a link is connected to a column, the following additional requirements shall be met:

10.12.4.1: Links connected to columns shall not exceed the length of $1.6\,M_p/V_y$ unless it can be demonstrated that the link-to-column connection is adequate to develop the required inelastic rotation of the link.

10.12.4.2: The link flanges shall have complete penetration welded joints to the column. The connection of the link web to the column shall be welded to develop the design axial, shear and flexural strength of the link web.

10.12.4.3: Where the link is connected to the column web, the link flanges shall have complete penetration welded joints to connection plates and the web connection shall be welded to develop the design axial, shear and flexural strength of the link web. The link rotation angle shall not exceed 0.015 radians for any link length.

10.12.5 LATERAL SUPPORT OF LINK: Lateral supports shall be provided at both the top and bottom flanges of link at the ends of the link. End lateral supports of links shall have a design strength of 4 percent of the link flange nominal strength computed as $F_y b_f t_f$.

10.12.6 DIAGONAL BRACE AND BEAM OUTSIDE OF LINK:

10.12.6.1: The required axial and moment strengths of each diagonal brace and the beam outside of the link shall be the axial forces and moments generated by 1.5 times the design shear strength of the link as defined in Sec. 10.12.2. The nominal strengths of the diagonal brace and of the beam outside of the link shall exceed the required strengths as defined above.

10.12.6.2: Diagonal brace-to-link connections shall develop the nominal strength of the brace and transfer this force to the beam. No part of the brace-to-beam connection shall extend over the link length. If the brace resists a portion of the link end moment as described above, the brace-to-beam connection shall be designed as fully restrained (Type FR) connection.

10.12.6.3: The beam outside of the link 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 design shear strength of the link. Lateral supports shall be provided at both top and bottom flanges of the beam and shall have a strength to resist at least 1.5 percent of the beam flange nominal strength computed as $F_y b_f t_f$.

10.12.6.4: The width-thickness ratio of brace shall satisfy Sec. 10.11.2.4.

10.12.7 BEAM-TO-COLUMN CONNECTIONS: Beam-to-column connections away from links are permitted to be designed as a pin in the plane of the web. The connection shall have a strength to resist rotation about the longitudinal axis of the beam based on two equal and opposite rotational forces of at least 1.5 percent of the beam flange nominal strength computed as $F_{by} b_f t_{bf}$ acting laterally on the beam flanges.

10.12.8 REQUIRED COLUMN STRENGTH: The required strength of columns shall be determined by Eq. 3-1 and 3-2 except that the moments and axial loads introduced into the column at the connection of a link or brace shall not be less than those generated by 1.25 times the design strength of the link.

10.13 LIGHT FRAMED WALL REQUIREMENTS:

10.13.1 SCOPE: Cold-formed steel light framed wall (stud wall) systems shall be designed in accordance with Ref. 10.3 and, when required, by the additional provisions of this section.

10.13.2 BOUNDARY MEMBERS: All boundary members, chords, and collectors shall be designed and detailed to transmit the induced axial force.

10.13.3 CONNECTIONS: Connections of diagonal bracing members, top chord splices, boundary members and collectors shall be designed to develop the tensile strength of the member or $(2R/5)$ equal to or greater than 1.0 times the design seismic forces. Pullout resistance of screws shall not be used to resist seismic forces.

10.13.4 BRACED BAY MEMBERS: Vertical and diagonal members of braced bays shall be anchored so the bottom track is not required to resist uplift force [load]s by bending of the track web. Both flanges of studs in a bracing bay shall be braced to prevent lateral torsional buckling. Wire tied bridging shall not be considered to provide such restraint.

10.14 ALLOWABLE STRESS DESIGN (ASD) ALTERNATIVE: As an alternative to the LRFD design procedures for structural steel design in Ref. 10.1, the ASD design procedures in Ref. 10.2 are permitted as modified by this section. When using Ref. 10.2, the provisions of Chapter 10 shall apply except the following sections shall be substituted for, or added to the appropriate sections as indicated:

1. ASD 10.2 Add the following at the end of Sec. 10.2:

The design strengths of structural steel members and connections subjected to seismic forces in combination with other prescribed loads shall be determined by converting allowable stresses into nominal strengths, as required by this section, and multiplying such nominal strengths by resistance factors herein.

Resistance factors (ϕ) for use with Ref. 10.2 shall be as follows:

Flexure	ϕ_b = 0.90
Compression and axially loaded composite members	ϕ_c = 0.85
Eyebars and pin connected members	
Shear on the effective area	ϕ_{sf} = 0.75
Tension on net effective area	ϕ_t = 0.75
Bearing on the project area of pin	ϕ_t = 1.0
Tension members	
Yielding in the gross section	ϕ_t = 0.90
Fracture in the net section	ϕ_t = 0.75
Shear	ϕ_v = 0.90
Connectors	
Welded connections, and base plates that develop the strength of the members or structural systems	ϕ = 0.90
Welded connections that do not develop the strength of the member or structural system, including connection of base plates and anchor bolts	ϕ = 0.67
Metal deck diaphragms	ϕ = 0.60
Partial penetration welds in columns when subjected to tension stresses	ϕ = 0.80
High strength bolts (A325 and A490) and rivets (A502)	
Tensile strength	ϕ = 0.75
Shear strength in bearing-type connections	ϕ = 0.65

Slip-critical joints $\phi = 1.0$

A307 bolts

Tensile strength $\phi = 0.75$

Shear strength in bearing-type connections $\phi = 0.60$

ASD 10.2.3 Structural Steel: Ref. 10.2 shall be modified as follows:

ASD 10.2.3.1 Nominal Strength: Sec. A5.2 shall read as follows:

> "The nominal 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 Sec. D, E, F, and J."

Amend the first paragraph of Sec. N1 by deleting "or earthquake" and adding the following:

> "The nominal strength of members shall be determined by the requirements contained herein. Except as modified by these rules, all pertinent provisions of Chapters A through M shall govern."

ASD 10.2.3.2 Euler Stress: In Sec. H1, for the purpose of determining the nominal strength of structural steel members, the definition of F_e, shall read as follows:

$$F_e' = \frac{\pi^2 E}{\left(\frac{kl_b}{r_b}\right)^2} \qquad (10\text{-}12)$$

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.

2. **Substitute the following for Sec. 10.2.2.3:**

ASD 10.2.2.3 Steel Deck Diaphragms: Nominal strength values of steel deck diaphragms, made from materials conforming to the requirements of Ref. 10.3, and 10.4, are permitted to be assigned in accordance with one of the following:

1. The tested strength values as approved by the regulatory agency,

2. 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 nominal strengths.

3. Substitute the following for Sec. 10.9:

ASD 10.9 ORDINARY MOMENT FRAME (OMF) REQUIREMENTS:

ASD 10.9.1 Design Strength: Ordinary moment frames (OMFs) shall have the design strength to resist the factored load combinations in Eq. 3-1 and 3-2. The design strength of such members shall be determined using Ref. 10.2 as modified by this chapter.

ASD 10.9.2 Joint Requirements: All beam-to-column connections in OMFs which resist earthquake forces shall meet one of the following requirements:

1. Sec. 10.10.2

2. The design shall be capable of inelastic deformations and the required strengths of the connections shall meet the requirements of Sec. 10.9.1 using the load combinations Eq. 3-1a and Eq. 3-2a.

3. Either Type 1 or Type 3 connections are permitted provided:

   a. The design strengths of the members and connections meet the requirements of Sec. 10.9.1,

   b. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at an elastic story drift calculated at ($2R/5$) greater than or equal to 1.0 times the design seismic forces, and

   c. The additional drift due to Type 3 connections shall be considered in design.

4. Substitute the following for Sec. 10.10.3.3:

ASD 10.10.3.3 Panel Zone Doubler Plates: Doubler plates provided to increase the design strength of the panel zone or to reduce the web depth thickness ratio shall be placed close to the column web and welded across the plate width top and bottom with a minimum fillet weld per Table J2.4 of Ref. 10.2. The doubler plates shall be fastened to the column flanges using either butt or fillet welded joints to develop the design shear strength of the doubler plate.

5. Substitute the following for Sec. 10.10.4.2:

ASD 10.10.4.2 Width-Thickness Ratios: The width-thickness ratios of compression elements of beams shall comply with λ_p in Table ASD 10.10.4.2 in lieu of those in Table B5.1 of Ref. 10.2.

ASD TABLE 10.10.4.2
Limiting Width Thickness Ratios λ_p for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios λ_p
Flanges of I-shaped non-hybrid sections and channels in flexure Flanges of I-shaped hybrid beams in flexure	b/t	$\dfrac{52}{\sqrt{F_y}}$
Webs in combined flexural and axial compression	h_c/t_w	For $P_u/\phi_b P_y \leq 0.125$ $\dfrac{520}{\sqrt{F_y}}\left(1 - \dfrac{1.54P_u}{\phi_b P_y}\right)$ For $P_u/\phi_b P_y > 0.125$ $\dfrac{191}{\sqrt{F_y}}\left(2.33 - \dfrac{P_u}{\phi_b P_y}\right) \geq \dfrac{253}{\sqrt{F_y}}$

6. Substitute the following for Sec. 10.10.7.2:

ASD 10.10.7.2 Unrestrained Connections: A column containing a beam-to-column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height and conform to Sec. B4 and H1 of Ref. 10.2 except that:

1. The required column strength shall be determined from the Eq. 3-1 where Q_E is the least of:

    a. The earthquake force amplified by $(2R/5)$ equal to or greater than 1.0.

b. 125 percent of the frame design strength based on either beam or panel zone design strengths.

2. The nominal column axial strength (P_n) shall be based on a pin ended column.

3. The L/r for these columns shall not exceed 60.

4. The required column moment (M_{uy}) shall include that caused by the beam flange force specified in Sec. 10.10.7.1.2.c plus that added P-delta moment due to the resulting column flange displacement.

7. Substitute the following for Sec. 10.12.6.2:

ASD 10.12.6.2: Diagonal brace-to-link connections shall develop the nominal strength of the brace and transfer this force to the beam. No part of the brace-to-beam connection shall extend over the link length. If the brace resists a portion of the link end moment as described above, the brace-to-beam connection shall be designed as a Type 1 connection.

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 318-89, excluding Appendix A

11.1.1 MODIFICATIONS TO REF. 11.1:

11.1.1.1: Replace Sec. 9.2.3 with Sec. 3.7 of this document.

11.1.1.2: Replace Sec. 21.2.1.3 and 21.2.1.4 with the provisions of this chapter.

11.1.1.3: Amend Sec. 21.2.1.5 to read as follows:

> "A reinforced concrete structural system not satisfying the requirements of this chapter, including those composed of precast elements, shall be permitted 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 chapter."

11.1.1.4: Add the following at the end of Sec. 21.2.5.1:

> "Post-tensioning tendons shall be permitted 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."

11.1.1.5: Insert the following new Sec. 21.3.2.3 and change the existing Sec. 21.3.2.3 and 21.3.2.4 to 21. 3.2.4 and 21.3.2.5, 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. Anchorage 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. 21.3.3.4 to read as follows:

"Where hoops are not required, stirrups with 135-degree or greater hooks with 6-bar-diameter but not less than 3-inch 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. 21.4.4:

"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. 21.7 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: Add the following to the end of Sec. 21.5.1:

"A cast-in-place topping on a precast floor system shall be permitted to 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 be 0.7."

11.1.1.9:* Add the following new Sec. 21.5.4 as follows and renumber existing Sec. 21.5.4 and 21.5.5 to 21.5.5 and 21.5.6, respectively:

21.5.4 Coupling Beams: Coupling beams interconnecting shear walls and with clear-span-to-effective-depth ratio (l_n/d) of less than four and with factored shear force V_u exceeding $4\sqrt{f'_c}\, b_w d$ shall be provided with shear reinforcement as specified in Sec. 21.5.4.1 through 21.5.4.3.

21.5.4.1: Factored shear force V_u shall be resisted by two intersecting groups of symmetrical diagonally placed bars extending across the full length of the member and adequately anchored within the shear walls. Each group shall consist of a minimum of four bars providing an area A_{vd} not less than that given by Eq. 21-a:

$$A_{vd} = \frac{V_u}{2 f_y \sin \alpha}$$

where α is the angle between diagonal reinforcement and longitudinal axis of the member.

21.5.4.2: Contribution of the diagonal reinforcement to nominal flexural strength of the coupling beam area shall be considered.

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*The addition of this section was approved by the required two-thirds majority of BSSC members during balloting; however, one negative voter pointed out that there is an apparent inconsistency with the original research and that perhaps a ϕ factor of 0.6 should be included in the equation for A_{vd}. Sufficient time was not available before printing of the 1991 Edition to permit resolution of this negative vote. The matter will be given attention during the next updating of the *Provisions*.

21.5.4.3: Each group of diagonally placed bars shall be enclosed in transverse reinforcement conforming to Sec. 21.4.4.1 through 21.4.4.3. For the purpose of computing A_g per Eq. 10-5 and 21-3, minimum cover as specified in Sec. 7.7 shall be assumed over each group of diagonally placed reinforcing bars.

11.1.1.10: Change the reference to Sec. 9.2 in Sec. 21.7.1.3 to the load combination specified in Sec. 3.7 of this document for earthquake forces.

11.1.1.11: Change the title of Sec. 21.9 to read: "Requirements for Intermediate Moment Frames."

11.2 BOLTS AND HEADED STUD ANCHORS IN CONCRETE: Bolts and headed stud anchors shall be solidly cast in concrete. The factored loads on embedded anchor bolts and headed stud anchors shall not exceed the design strengths determined by Sec. 11.2.2.

11.2.1 LOAD FACTOR MULTIPLIERS: In addition to the load factors in Sec. 3.7, a multiplier of 2 shall be used if special inspection is not provided or of 1.3 if it is provided. When anchors are embedded in the tension zone of a member, the load factors in Sec. 3.7 shall have a multiplier of 3 if special inspection is not provided or of 2 if it is provided.

11.2.2 STRENGTH OF ANCHORS: The strength of headed bolts and headed studs solidly cast in concrete shall be taken as the average of 10 tests for each concrete strength and anchor size or calculated as the minimum of P_s or ϕP_c in tension and V_s or ϕV_c in shear when:

$$P_s = 0.9 A_b f_s'$$

and

$$\phi P_c = \lambda \sqrt{f_c'}\,(2.8 A_s + 4 A_t)$$

where

A_b = Area (in square inches) of bolt or stud. Must be used with the corresponding steel properties to determine the weakest part of the assembly in tension. In shear, the insert leg need not be checked.

A_s = The sloping area (in square inches) of an assumed failure surface. The surface to be that of a cone or truncated pyramid radiating at a 45 degree slope from the bearing edge of the anchor or anchor group to the surface. For thin sections with anchor groups, the failure surface shall be assumed to follow the extension of this slope through to the far side rather than truncate as in A_t.

A_t = The area (in square inches) of the flat bottom of the truncated pyramid of an assumed concrete failure surface. When anchors in a group are closer together than twice their embedment length, the failure surface pyramid is assumed to truncate at the anchor bearing edge rather than form separate cones.

f'_c = Concrete strength, 6,000 psi maximum for design.

f'_s = Ultimate tensile strength (in psi) of the bolt, stud, or insert leg wires not to be taken greater than 60,000 psi. For A307 bolts or A108 studs, may be assumed to be 60,000.

$P_u V_u$ = Tensile, shear strength required due to factored loads (in pounds).

λ = 1 for normal weight, 0.75 for "all lightweight," and 0.85 for "sand lightweight" concrete.

ϕ = Strength reduction factor = 0.65.

EXCEPTION: When the anchor is attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to reinforcing steel that is designed to distribute forces and avert sudden local failure, ϕ may be taken as 0.85.

When edge distance is less than embedment length, reduce proportionately. For multiple edge distances less than the embedment length, use multiple reductions:

$$V_s = 0.75 A_b f'_s$$

when loaded toward an edge greater than 10 diameters away,

$$\phi V_c = \phi 800 A_b \lambda \sqrt{f'_c}$$

when loaded toward an edge less than 10 diameters away,

$$\phi V_c = \phi 2 \pi d_e^2 \lambda \sqrt{f'_c}$$

where d_e = distance from the anchor axis to the free edge.

For groups of anchors, the concrete design shear strength shall be taken as the smallest of:

1. The strength of the weakest stud times the number of studs,
2. The strength of the row of studs nearest the free edge in the direction of shear times the number of rows, or
3. The strength of the row farthest from the free edge in the direction of shear.

For shear loading toward an edge less than 10 diameters away, or tension or shear not toward an edge less than 4 diameters away, reinforcing sufficient to carry the load shall be provided to prevent failure of the concrete in tension. In no case shall the edge distance be less than one-third the above. The bearing area of headed anchors shall be at least one and one-half times the shank area for anchors of not over 60,000 psi yield strength.

When tension and shear act simultaneously, both the following shall be met:

$$\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right]^{1/2} \leq 1$$

and

$$\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \leq 1$$

11.2.3 ANCHOR BOLTS IN TOPS OF COLUMNS: Anchor bolts at the top of columns shall have a minimum embedment of 9 bolt diameters and shall be enclosed with not less than two No. 4 ties located within 4 inches of the column top.

11.3 CLASSIFICATION OF MOMENT FRAMES:

11.3.1 ORDINARY MOMENT FRAMES: Ordinary moment frames are frames conforming to the requirements of Ref. 11.1 exclusive of Chapter 21.

11.3.2 INTERMEDIATE MOMENT FRAMES: Intermediate moment frames are frames conforming to the requirements of Sec. 21.9. of Ref. 11.1 in addition to those requirements for ordinary moment frames.

11.3.3 SPECIAL MOMENT FRAMES: Special moment frames are frames conforming to the requirements of Sec. 21.2 through 21.4, 21.6, and 21.7 of Ref. 11.1 in addition to those requirements for ordinary moment frames.

11.4 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A may be of any construction permitted in Ref. 11.1 and these provisions.

11.5 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.

11.6 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.

11.6.1 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system shall be intermediate moment frames conforming to Sec. 11.3.2 or special moment frames conforming to Sec. 11.3.3.

11.6.2 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. 21.9.5.1 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 required in Sec. 21.4.4.5 of Ref. 11.1.

11.7 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.

11.7.1 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system, regardless of height, shall be special moment frames conforming to Sec. 11.3.3.

11.7.2 SEISMIC FORCE RESISTING SYSTEM: All materials and components in the seismic force resisting system shall conform to Sec. 21.2 through 21.7 of Ref. 11.1.

11.7.3 FRAME MEMBERS NOT PROPORTIONED TO RESIST FORCES INDUCED BY EARTHQUAKE MOTIONS: All frame components assumed not to contribute to lateral force resistance shall conform to Sec. 3.3.4.3 of these provisions and to Sec. 21.8.1.1 or 21.8.1.2 and 28.8.2 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 530-88/ASCE 5-88, including Appendix A, Special Provisions for Seismic Design, and Specifications for Masonry Structures, ACI 530.1-88/ASCE 6-88.

12.1.1 MODIFICATIONS TO APPENDIX A OF REFERENCE 12.1:

12.1.1.1: Replace all references to seismic zones (ANSI A58.1 zones) with the Seismic Performance Categories listed in Table 12.1.1.

TABLE 12.1.1
ANSI 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

12.1.1.2: The requirements of Sec. 3.7 shall apply in lieu of the load and load combination provisions of Ref. 12.1, Chapter 5.

12.1.1.3: The requirements of Ref. 12.1, Sec. A.3.3, shall not apply.

12.1.1.4: The requirements of Ref. 12.1, Sec. A.3.4, shall not apply.

12.1.1.5: The requirements of Sec. 3.6.1.2 shall apply in lieu of the requirements of Ref. 12.1, Sec. A.3.6.

12.1.1.6: The requirements of Ref. 12.1, Sec. A.4.9.1, shall not apply.

12.1.1.7: The maximum spacing of reinforcement requirements of Sec. 12.7.2.1 shall apply in lieu of those in Ref. 12.1, Sec. A.4.9.1.2.

12.2 STRENGTH OF MEMBERS AND CONNECTIONS: The design strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor (ϕ) and 2.5 times the allowable working 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 response modification factors (R) of Table 3.3 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 3.3 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 B; 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 per nominal 4 inches of wythe or less 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 Load-bearing Wall Tile (ASTM C 34)
Structural Clay Nonload-bearing 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.

APPENDIX TO CHAPTER 12

Masonry Limit States Design Provisions

PREFACE: The masonry provisions in the body of the 1991 Edition of the Provisions are based on working stress design criteria modified by a factor to simulate strength design criteria. It is recognized that the simulated strength design criteria are not state of the art because they do not predict the performance of masonry beyond the elastic range. Research conducted to date documents the behavior of masonry both at and beyond peak strength levels and the limit states design provisions in this appendix have been developed based on this documented laboratory performance of masonry. This appendix is included for information and to provide a better understanding of masonry performance under seismic loads. It is considered premature to base code provisions on this resource appendix; however, user review, trial design, and comment on this appendix are encouraged. Please direct such feedback to the BSSC.

12A.1 GENERAL:

12A.1.1 SCOPE: Requirements for the design and construction of reinforced and plain masonry components and systems and for the materials used therein shall comply with the requirements of this chapter.

12A.1.2 REFERENCE DOCUMENTS: The designation and title of documents cited in this appendix are listed in this section. Compliance with specific provisions of these reference documents is mandatory where required by this chapter.

ACI 530-88/ASCE 5-88	Building Code Requirements for Masonry Structures
ACI 530.1-88/ASCE 6-88	Specifications for Masonry Structures
AWS D1.4-79	Structural Welding Code--Reinforcing Steel

12A.1.3 DEFINITIONS:

Anchor: Metal rod , wire, or strap that secures masonry to its structural support.

Area:

Gross Cross-Sectional Area: The area within the perimeter of masonry in the plane under consideration.

Net Cross-Sectional Area: The area of masonry units, grout, and mortar crossed by the plane under consideration.

Bed Joint: The horizontal layer of mortar on which a masonry unit is laid.

Cleanout: An opening to the bottom of a grout space of sufficient size and spacing to allow removal of debris.

Collar Joint: Vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction that may be filled with mortar or grout.

Column: An isolated vertical member whose horizontal dimension measured at right angles to the thickness does not exceed three times its thickness and whose height is at least six times its thickness.

Composite Action: Transfer of stress between components of a member designed so that the combined components act together as a single member in resisting loads.

Composite Masonry: Multi-wythe masonry members acting with composite action.

Compressive Strength of Masonry: Maximum compressive force resisted per unit of net cross-sectional area of masonry.

Connector: A mechanical device for securing two or more pieces, parts, or members together including anchors, wall ties, and fasteners.

Dimension

> **Actual Dimension:** The measured dimension of a designated item (e.g., a designated masonry unit or wall) as used in the structure.
>
> **Nominal Dimension:** The dimension equal to specified dimensions plus the thickness of the joint with which the unit is laid.
>
> **Specified Dimension:** The dimension specified for the manufacture or construction of masonry, masonry units, joints, or any other component of a structure. Unless otherwise stated, all calculations shall be based on specified dimensions.

Effective Height: Clear height of member between support points.

Grout Lift: An increment of grout height within the total grout pour; a grout pour may consist of one or more grout lifts.

Grout Pour: The total height of masonry wall to be filled with grout prior to the erection of additional masonry; a grout pour consists of one or more grout lifts.

Head Joint: Vertical mortar joint placed between masonry units within the wythe at the time the masonry units are laid.

Masonry Unit:

Hollow Masonry Unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75 percent of the gross cross-sectional area in the same plane.

Solid Masonry Unit: A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.

Plastic Hinge: The yield zone that forms in a structural member subjected to a moment exceeding the yield moment.

Prism, Masonry: An assemblage of masonry units and mortar with or without grout used as a test specimen for determining compliance with f'_m.

Reinforced Masonry: Masonry construction in which reinforcement is calculated to act in conjunction with the masonry to resist forces.

Running Bond: The placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length.

Specified Compressive Strength of Masonry (f'_m): Required compressive strength (expressed as force per unit of net cross-sectional area) of the masonry. Whenever the quantity f'_m is under the radical sign, the square root of numerical value only is intended and the result has units of pounds per square inch.

Stack Bond: For the purpose of this appendix, stack bond is other than running bond. Usually, the placement of units is such that the head joints in successive courses are vertically aligned.

Stone Masonry: Masonry composed of field, quarried, or cast stone units bonded by mortar.

Ashlar Stone Masonry: Stone masonry composed of rectangular units having sawed, dressed, or squared bed surfaces and bonded by mortar.

Rubble Stone Masonry: Stone masonry composed or irregular shaped units bonded by mortar.

Strength

Design Strength: Nominal strength multiplied by a strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross section calculated in accordance with these limit state design provisions before application of any strength reduction factors.

Required Strength: Strength of a member or cross section required to resist factored loads.

Tie

Confinement Tie: Steel reinforcement such as a loop of reinforcing bar, masonry comb, or steel plate that is embedded in a masonry section normal to the design applied force for the purpose of increasing usable compressive strain in the masonry.

Lateral Tie: Loop of reinforcement enclosing longitudinal reinforcement.

Wall Tie: Metal connector that connects wythes of masonry walls together.

Wall: A vertical element with a horizontal length at least three times its thickness.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

12A.1.4 NOTATIONS:

A_b = cross-sectional area of an anchor bolt, in.^{2}

A_g = gross cross-sectional area of masonry, in.^{2}

A_{mv} = net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered, in.^{2}

A_n = net cross-sectional area of masonry, in.^{2}

A_p = projected area, on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations, in.^{2}

A_s = cross-sectional area of reinforcement, in.^{2}

a_b = length of compressive stress block, inches

B_a = nominal axial strength of an anchor bolt, lb

B_v = nominal shear strength of an anchor bolt, lb

b = effective width of a member, inches

b_a = design axial force on an anchor bolt, lb

b_v = design shear force on an anchor bolt, lb

b_w = web width, inches

d = depth of member, inches

d_b = diameter of reinforcement, inches

E_m = chord modulus of elasticity of masonry, psi

E_s = modulus of elasticity of reinforcement, psi

E_v = shear modulus of masonry, psi

e = eccentricity of P_u, inches

f'_m = specified compressive strength of masonry at the age of 28 days, unless other age is specified, psi

f_r = modulus of rupture of masonry, psi

f_y = specified yield strength of reinforcement, psi

h = effective height of the wall between points of support, inches

I_{cr} = moment of inertia of the cracked section, in.^{4}

I_n = moment of inertia of the net cross-sectional area of a member, in.^{4}

L = length of coupling beam between coupled shear walls

ℓ_b = effective embedment length of anchor bolt, inches

ℓ_{be} = anchor bolt edge distance, inches

ℓ_d = development length, inches

ℓ_{dh} = equivalent development length for a standard hook, inches

ℓ_{ld} = minimum lap splice length, inches

ℓ_u = unsupported length of compression member, inches

ℓ_w = length of wall, inches

M = moment on a masonry section due to unfactored design load, in.-lb

M_a = maximum moment in member at stage deflection is computed, in.-lb

M_{cr} = cracking moment strength of the masonry, in.-lb

M_d	=	design moment strength, in.-lb
M_u	=	required moment strength, in.-lb
M_1	=	axial load acting concurrently with shear force, lb
M_2	=	nominal moment strength at the ends of the coupling beam, lb
N_v	=	force acting normal to shear surface, lb
P	=	axial load on a masonry section due to unfactored loads, lb
P_n	=	nominal axial load strength, lb
P_u	=	required axial load strength, lb
r	=	radius of gyration, inches
S	=	section modulus, in.^{3}
s	=	spacing of shear reinforcement, inches
t	=	wall thickness or least lateral dimensions of column, inches
U	=	required strength to resist factored loads, or related internal moments and forces
V	=	shear on a masonry section due to unfactored design loads, lb
V_m	=	shear strength of masonry, lb
V_n	=	nominal shear strength, lb
V_s	=	shear strength provided by shear reinforcement, lb
V_u	=	required shear strength, lb
Δ	=	computed deflection for serviceability, inches
Δ_{cr}	=	computed deflection at cracking moment strength level, inches
ρ	=	ratio of the area of reinforcement to the cross sectional area of masonry on a plane perpendicular to the reinforcement
ρ_b	=	reinforcement ratio producing balanced strain conditions

ρ_h = ratio of the area of shear reinforcement to the cross sectional area of masonry on a plane perpendicular to the reinforcement

ϵ_{mu} = maximum usable compressive strain of masonry, in./in.

ϕ = strength reduction factor

12A.2 CONSTRUCTION REQUIREMENTS:

12A.2.1 GENERAL: Masonry shall be constructed in accordance with the requirements of ACI 530.1/ASCE 6 including the requirements for masonry materials stated therein.

12A.2.2 QUALITY ASSURANCE: Inspection and testing of masonry materials and construction shall comply with the requirements of Sec. 1.6.

12A.3 GENERAL DESIGN REQUIREMENTS:

12A.3.1 SCOPE: Masonry structures and components of masonry structures shall be designed in accordance with the requirements of reinforced masonry design, plain masonry design, or empirical design subject to the limitations of this section.

12A.3.2 EMPIRICAL MASONRY DESIGN: The requirements of ACI 530/ASCE 5, Chapter 9, shall apply to the design of empirical masonry.

12A.3.3 PLAIN MASONRY DESIGN:

12A.3.3.1: In the design of plain masonry members, the flexural tensile strength of masonry units, mortar, and grout in resisting design loads shall be permitted.

12A.3.3.2: In the design of plain masonry members, stresses in reinforcement shall not be considered effective in resisting design loads.

12A.3.3.3: Plain masonry members shall be designed to remain uncracked.

12A.3.3.4: The response modification factors (R) of Table 3.3 for unreinforced masonry shall apply to plain masonry.

12A.3.4 REINFORCED MASONRY DESIGN: In the design of reinforced masonry members, stresses in reinforcement shall be considered effective in resisting design loads.

12A.3.5 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A shall comply with the requirements of Sec. 12A.3.2. 12A.3.3, or 12A.3.4.

12A.3.6 SEISMIC PERFORMANCE CATEGORY B: Buildings assigned to Category B shall conform to all the requirements for Category A and the lateral force resisting system shall be designed in accordance with Sec. 12A.3.3 or Sec. 12A.3.4.

12A.3.7 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C shall conform to the requirements for Category B and to the additional requirements of this section.

12A.3.7.1: Veneer and units not specifically intended for structural use shall not be designed to resist loads other than their own weight or their own shear loads.

12A.3.7.2: Masonry walls shall be anchored to all floors and roofs that provide lateral support for the walls. Such anchorage shall provide a direct connection capable of resisting horizontal design forces or a minimum of 200 pounds per lineal foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where anchor spacing exceeds 4 feet. Anchors in masonry walls shall be embedded in reinforced bond beams or reinforced vertical cells.

12A.3.7.3: Structural members framing into or supported by masonry columns shall be anchored thereto. Anchor bolts located in the tops of columns shall be set entirely within the reinforcing cage composed of column bars and lateral ties. A minimum of two No. 4 lateral ties shall be provided in the top 5 inches of the column.

12A.3.7.4: Vertical reinforcement of at least 0.20 square inch in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, and at the ends of walls. Horizontal reinforcement not less than 0.2 square inch in cross section shall be provided as follows:

a. At the bottom and top of wall openings extending not less than 24 inches or less than 40 bar diameters past the opening,

b. Continuously at structurally connected roof and floor levels and at the top of walls,

c. At the bottom of load-bearing walls or in the top of foundations when dowelled to the wall,

d. At maximum spacing of 10 feet unless uniformly distributed joint reinforcement is provided.

Reinforcement at the top and bottom of openings, when used in determining the maximum spacing specified in Item d above, shall be continuous in the wall.

12A.3.7.5: Where head joints in successive courses are horizontally offset less than one-quarter of the unit length, the minimum horizontal reinforcement shall be 0.0007 times the gross cross-sectional area of the wall. This reinforcement shall be satisfied with uniformly distributed joint reinforcement or with horizontal reinforcement spaced not over 4 feet and fully embedded in grout or mortar.

12A.3.7.6 Multiple Wythe Walls Not Acting Compositely: At least one wythe of a cavity wall shall be designed and reinforced in accordance with Sec. 12A.3.4; the other wythe shall be tied to its back-up and reinforced with a minimum of one No. 9 wire gage per nominal 4 inches of wythe or less at a maximum spacing of 16 inches o.c. The wythes shall be tied in accordance with ACI 530-88/ASCE 5-88, Sec. 5.8.2.2.

12A.3.7.7 Screen Walls: Masonry screen walls, laterally supported but not otherwise connected on all edges by a structural frame of masonry, concrete, or steel, shall meet the following requirements:

12A.3.7.7.1: 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.

12A.3.7.7.2: 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.

12A.3.7.8 Material Requirements: The following materials shall not be used for any structural masonry:

Structural Clay Load-bearing Wall Tile (ASTM C 34)
Structural Clay Nonload-bearing Wall Tile (ASTM C 56)

12A.3.8 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.

12A.3.8.1: Masonry shall not be used for shear walls to resist seismic loading in buildings over 160 feet in height unless such buildings employ ductile moment resisting space frames capable of resisting at least 25 percent of seismic loading.

12A.3.8.2: The following materials shall not be used as part of the structural system:

Type N mortar
Masonry cement

12A.3.8.3: All walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in each direction shall not be less than 0.0007 times the gross cross-sectional area of the wall. The spacing of reinforcement shall not exceed 4 feet except the spacing of principal reinforcement shall not exceed 2 feet for the buildings using a moment resisting space frame. The diameter of reinforcement shall not be less than 3/8 inch except for joint reinforcement. Reinforcement shall be continuous around wall corners and through intersections. Only horizontal reinforcement that is continuous in the wall or element shall be included in computing the area of reinforcement. Reinforcement spliced in accordance with Sec. 12A.4.5.6 shall be considered as continuous reinforcement.

12A.3.8.4: The spacing of column ties shall not be more than 8 inches for the full height of columns stressed by tensile or compressive axial overturning forces due to seismic loads. For all other columns, the spacing of ties shall not exceed 8 inches over a distance not less than one-sixth of the column height at the top and bottom of the column, 18 inches, or the maximum column side dimension. Lateral tie spacing for the remaining column height shall be not more than 16 bar diameters, 48 tie diameters, the least column dimension, or 18 inches. Lateral column ties shall be No. 3 or larger.

Lateral ties for compression reinforcement shall be embedded in grout.

12A.3.8.5: Where masonry is laid in other than running bond, the minimum amount of horizontal reinforcement shall be 0.0015 times the gross cross-sectional area of the wall. If open-end units are used and grouted solid, the minimum amount of horizontal reinforcement shall be 0.0007 times the gross cross-sectional area of the wall.

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

12A.3.8.6 Minimum Dimensions:

12A.3.8.6.1: The nominal thickness of masonry bearing walls shall not be less than 6 inches. Nominal 4-inch-thick load-bearing reinforced hollow clay unit masonry walls with a maximum unsupported height or length to thickness of 27 are permitted to be used provided the net area unit strength exceeds 8,000 psi, units are laid in running bond, bar sizes do not exceed 1/2 inch with not more than two bars or one splice in a cell, and joints are not raked.

12A.3.8.6.2: The least nominal dimensions of a masonry column shall be 12 inches.

12A.3.8.7 Shear Walls:

12A.3.8.7.1: Reinforcement required to resist in-plane shear shall be uniformly distributed and shall be embedded in mortar or grout. 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. Reinforcement shall be placed horizontally.

12A.3.8.7.2: Reinforcement required to resist in-plane shear shall be terminated with a standard hook or anchored by embedment beyond the end of the wall. The hook or extension shall be anchored up, down, or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

12A.3.8.8 Concrete abutting structural masonry as at starter courses not designed as true separation joints shall be roughened to a full amplitude of 1/16 inch and shall be bonded to the masonry in accordance with these provisions as if it were masonry. Vertical joints not designed as true separation joints shall be reinforced as required for other than running bond in Sec. 12A.3.8.5 and such reinforcement shall extend through the joint and be anchored to the concrete.

12A.3.9 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.

12A.3.9.1 Construction Requirements: Construction procedures or admixtures shall be used to minimize cracking of grout and to maximize bond.

12A.3.9.2 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.

12A.3.9.3 Stacked Bond Construction: All stacked bond construction shall conform to the following requirements:

12A.3.9.3.1: 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.

12A.3.9.3.2: 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.

12A.3.9.3.3: 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.

12A.3.10: MATERIALS PROPERTIES:

12A.3.10.1: Modulus of elasticity of reinforcement for design, unless otherwise determined by test, shall be based on a steel reinforcement modulus of elasticity (E_s) equal to 29,000,000 psi.

12A.3.10.2: The modulus of elasticity of masonry for design shall be based on the modulus of elasticity of masonry (E_m) determined in accordance with Table 12A.3.10.2 or shall be based on the chord modulus of elasticity determined by prism test and taken between 0.05 and 0.33 times the masonry prism strength.

TABLE 12A.3.10.2
Modulus of Elasticity [a] (psi x 10^6)

Net Area Compressive Strength of Units (psi)	Clay Masonry		
	Type N Mortar	Type S Mortar	Type M Mortar
12,000 and greater	2.8	3.0	3.0
10,000	2.4	2.9	3.0
8,000	2.0	2.4	2.8
6,000	1.6	1.9	2.2
4,000	1.2	1.4	1.6
2,000	0.8	0.9	1.0

Net Area Compressive Strength of Units (psi)	Concrete Masonry	
	Type N Mortar	Type M or S Mortar
6,000 or greater	---	3.5
5,000	2.8	3.2
4,000	2.6	2.9
3,000	2.3	2.5
2,500	2.2	2.4
2,000	1.8	2.2
1,500	1.5	1.6

[a] Linear interpolation permitted.

12A.3.10.3: Shear modulus of masonry for design shall be based on a shear modulus of masonry (E_v) equal to 0.4 times the modulus of elasticity of masonry (E_m).

12A.3.10.4 Masonry Compressive Strength:

12A.3.10.4.1: The specified compressive strength of masonry (f'_m) shall equal or exceed 1,500 psi.

12A.3.10.4.2: The value of f'_m used to determine nominal strength values in this chapter shall not be greater than 4,000 psi for concrete masonry and shall not be greater than 6,000 psi for clay masonry.

12A.3.10.5 Modulus of Rupture:

12A.3.10.5.1: Running Bond Masonry: The modulus of rupture (f_r) for masonry laid in running bond shall be taken from Table 12A.3.10.5.1.

TABLE 12A.3.10.5.1
Modulus of Rupture for Masonry Laid in Running Bond (f_r)

	Normal to Bed Joints (psi)	Normal to Head Joints (psi)
Solid units	90	160
Hollow units	50	100
Hollow units grouted solid	133	167

12A.3.10.5.2 Stack Bond Masonry: For grouted hollow unit stack bond masonry, the modulus of rupture (f_r) normal to the head joints shall be taken as 250 psi and shall be based on the grout cross section only. For ungrouted stack bond masonry, f_r shall be zero.

12A.3.10.6 Reinforcement Strength: Masonry design shall be based on a reinforcement strength value equal to the specified yield strength of reinforcement (f_y) that shall not exceed 60,000 psi.

12A.3.11 SECTION PROPERTIES:

12A.3.11.1: Member strength shall be computed using section properties based on the net cross sectional area of the member under consideration.

12A.3.11.2: Section properties shall be based on specified dimensions.

12A.3.12 PLATE, HEADED, AND BENT BAR ANCHOR BOLTS:

12A.3.12.1: The design axial strength for plate anchors, headed anchor bolts, and bent bar anchor bolts (J or L) embedded in masonry (B_a) shall be the lesser of Eq. 12A-1 or Eq. 12A-2:

$$B_a \le (\phi)\, 0.6 A_p \sqrt{f'_m} \qquad \text{(12A-1)}$$

or

$$B_a \le (\phi)\, 0.25 A_b \qquad \text{(12A-2)}$$

where

ϕ = 0.8, strength reduction factor,

A_p = projected area on the masonry surface of a right circular cone (in.^{2}),

A_b = cross-sectional area of the anchor bolt (in.^{2}), and

f'_m = compressive strength of the masonry.

12A.3.12.1.1: The area (A_p) shall be the lesser of Eq. 12A-3 or Eq. 12A-4:

$$A_p = \pi \ell_b^2 \qquad (12A\text{-}3)$$

$$A_p = \pi \ell_{be}^2 \qquad (12A\text{-}4)$$

where:

ℓ_b = effective embedment length of the anchor bolt (in.) and

ℓ_{be} = anchor bolt edge distance (in.).

Where the area (A_p) of adjacent anchor bolts overlap, the area (A_p) of each bolt shall be reduced by one-half of the overlapping area. That portion of the projected area falling in an open cell or core shall be deducted from the value of A_p calculated using Eq. 12A-3 or Eq. 12A-4.

12A.3.12.1.2: The effective embedment length of plate or headed bolts (ℓ_b) shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchor bolt.

12A.3.12.1.3: The effective embedment of bent anchors (ℓ_b) shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter.

12A.3.12.1.4: The minimum effective embedment length shall be 4 bolt diameters but at least 2 inches

12A.3.12.2: The allowable load in shear (B_v), where the anchor bolt edge distance (ℓ_{be}) equals or exceeds 12 bolt diameters, shall be the lesser of Eq. 12A-5 or Eq. 12A-6:

$$B_v = (\phi) 700 \sqrt[4]{f'_m A_b} \qquad (12A\text{-}5)$$

$$B_v = \phi 0.25 A_b f_y \qquad (12A\text{-}6)$$

where

ϕ = 0.5, strength reduction factor,

A_b = cross-sectional area of the anchor bolt (in.^{2}),

f'_m = compressive strength of the masonry (psi), and

f_y = yield strength of the reinforcement (psi).

Where the anchor bolt edge distance (ℓ_{be}) is less than 12 bolt diameters, the value of B_v in Eq. 12A-5 shall be reduced by linear interpolation to zero at an ℓ_{be} distance of 1 inch.

12A.3.12.3: Combined shear and tension. Anchors subjected to combined shear and tension shall be designed to satisfy the following equation:

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \leq 1 \tag{12A-7}$$

where

b_a = design axial force on the anchor,

B_a = nominal axial strength of the anchor,

b_v = design shear force on the anchor, and

B_v = nominal shear strength of the anchor.

12A.4 DETAILS OF REINFORCEMENT:

12A.4.1 GENERAL:

12A.4.1.1: Details of reinforcement shall be shown or covered by notes on the contract documents.

12A.4.1.2: Reinforcing bars shall be embedded in grout.

12A.4.2. SIZE OF REINFORCEMENT:

12A.4.2.1: The maximum size of reinforcing bars used in masonry shall be No. 9. The maximum area of reinforcing bars placed in hollow unit construction shall be 4 percent of the cell area.

12A.4.2.2: The diameter of reinforcing bars shall not exceed one-quarter the least clear dimension of the cell, bond beam, or collar joint in which it is placed. (See Sec. 12A.4.1.2.)

12A.4.2.3: Joint reinforcement shall conform to the following: Longitudinal and cross wires shall be minimum No. 11 gage (0.12 inch diameter) and not exceed one-half the joint thickness.

12A.4.3 PLACEMENT LIMITS FOR REINFORCEMENT:

12A.4.3.1: The clear distance between parallel reinforcing bars shall not be less than the nominal diameter of the bars, nor less than 1 inch.

12A.4.3.2: In columns and pilasters, the clear distance between vertical reinforcing bars shall not be less than one and one-half times the nominal bar diameter, nor less than 1-1/2 inches.

12A.4.3.3: The clear distance limitations between reinforcing bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

12A.4.3.4: Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to two in any one bundle. Individual bars in a bundle cut off within the span of a member shall terminate at points at least 40 bar diameters apart.

12A.4.3.5: Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than 1/2 inch for fine grout or 3/4 inch for coarse grout.

12A.4.4 PROTECTION FOR REINFORCEMENT:

12A.4.4.1: Reinforcing bars shall have a minimum masonry cover not less than the following:

a. Where the masonry face is exposed to earth or weather, 2 inches for bars larger than No. 5 and 1-1/2 inches for No. 5 bars or smaller.

b. Where the masonry not exposed to earth or weather, 1-1/2 inches.

12A.4.4.2: Longitudinal wires of joint reinforcement shall be fully embedded in mortar or grout with a minimum cover of 1/2 inch when exposed to earth or weather and 3/8 inch when not exposed to earth or weather. Joint reinforcement in masonry exposed to earth or weather shall be corrosion resistant or protected from corrosion by coating. (See Sec. 12A.4.1.1.)

12A.4.4.3: Wall ties, anchors, and inserts, except anchor bolts not exposed to the weather or moisture, shall be protected from corrosion.

12A.4.5 DEVELOPMENT OF REINFORCEMENT:

12A.4.5.1 GENERAL: The calculated tension or compression in the reinforcement where masonry reinforcing is anchored in concrete shall be developed in the concrete by embedment length, hook or mechanical device or a combination thereof. Hooks shall only be used to develop bars in tension.

12A.4.5.2 Embedment of Reinforcing Bars and Wires in Tension: The embedment length (ℓ_d) of bars and wire shall be determined by Eq. 12-8 but shall not be less than 12 inches for bars and 6 inches for wire:

$$\ell_d = \left(\frac{1}{\phi}\right) 0.0005 d_b f_y \qquad (12A\text{-}8)$$

where

ϕ = 0.8, strength reduction factor,

d_b = diameter of the reinforcement (in.), and

f_y = specified yield strength of the reinforcement (psi).

12A.4.5.3 Standard Hooks:

12A.4.5.3.1: The term standard hook as used in this code shall mean one of the following:

12A.4.5.3.1.1: A 180 degree turn plus extension of at least 4 bar diameters but not less than 2-1/2 inches at free end of bar.

12A.4.5.3.1.2: A 135 degree turn plus extension of at least 6 bar diameters at free end of bar.

12A.4.5.3.1.3: For stirrup and tie anchorage only, either a 135 degree or a 180 degree turn plus an extension of at least 6 bar diameters at the free end of the bar.

12A.4.5.4 Minimum Bend Diameter for Reinforcing Bars:

12A.4.5.4.1: The diameter of bend measured on the inside of the bar, other than for stirrups and ties, shall not be less than values specified in Table 12A.4.5.4.1.

TABLE 12A.4.5.4.1
Minimum Diameters of Bend

Bar size	Grade	Minimum Bend
No. 3 through No. 7	40	5 bar diameters
No. 3 through No. 8	50 or 60	6 bar diameters
No. 9	50 or 60	8 bar diameters

12A.4.5.4.2: The equivalent embedment length for standard hooks in tension (ℓ_{dh}) shall be as follows:

$$\ell_{dh} = \left(\frac{1}{\phi}\right) 13 d_b \qquad (12A\text{-}9)$$

where ϕ = 0.9, strength reduction factor, and d_b = diameter of the reinforcement (in.).

12A.4.5.4.3: The effect of hooks for bars in compression shall be neglected in design computations.

12A.4.5.5 Development of Shear Reinforcement:

12A.4.5.5.1 Bar and Wire Reinforcement:

12A.4.5.5.1.1: Shear reinforcement shall extend to a distance d from the extreme compression face and shall be carried as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit. Horizontal bars in shear walls shall be anchored by hooks around the vertical edge bars.

12A.4.5.5.1.2: The ends of single leg or U-stirrups shall be anchored by one of the following means:

a. A standard hook plus an effective embedment of 0.5 times the development length (ℓ_d). The effective embedment of a stirrup leg shall be taken as the distance between the mid-depth of the member ($d/2$) and the start of the hook (point of tangency).

b. For No. 5 bar and D31 wire, and smaller, bending around longitudinal reinforcement through at least 135 degrees plus an embedment of $\ell_d/3$. The $\ell_d/3$ embedment of a stirrup leg shall be taken as the distance between mid-depth of member ($d/2$) and the start of the hook (point of tangency).

c. Between the anchored end, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

12A.4.5.5.2 Wire Fabric:

12A.4.5.5.2.1: For each leg of welded wire fabric forming simple U-stirrups, there shall be either:

a. Two longitudinal wires spaced at a 2 inch spacing along the member at the top of the U or

b. One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2 inches from the first wire. The second wire shall be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

12A.4.5.5.2.2: For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced a minimum of 2 inches with the inner wire placed at a distance at least $d/4$ or 2 inches from mid-depth of member $d/2$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

12A.4.5.6 Splices of Reinforcement: Lap splices, welded splices, or mechanical splices may be used in accordance with the provisions of this section. All welding shall conform to AWS D1.4.

12A.4.5.6.1 Lap Splices:

12A.4.5.6.1.1: The minimum length of lap (ℓ_{ld}) for bars in tension or compression shall be determined by Eq. 12A-10 but shall not be less than 12 inches:

$$\ell_{ld} = \left(\frac{1}{\phi}\right)\left(\frac{0.0045\, d_b^2 f_y}{t - d_b}\right) \tag{12A-10}$$

where

ϕ = 0.9, strength reduction factor,

d_b = diameter of the reinforcement (in.),

f_y = specified yield strength of the reinforcement (psi), and

t = wall thickness (in.).

12A.4.5.6.1.2: Bars spliced by noncontact lap splices shall not be spaced transversely farther apart than one-fifth the required length of lap nor more than 8 inches.

12A.4.5.6.1.3: The minimum length of lap required in Eq. 12A-10 shall be reduced 25 percent when:

a. The area of reinforcement provided is at least twice that required by analysis over the entire length of the splice and

b. One-half or less of the total reinforcement is spliced within the required lap length.

12A.4.5.6.2 Welded Splices: A welded splice shall have the bars butted and welded to develop in tension 125 percent of the specified yield strength (f_y) of the bar.

12A.4.5.6.3 Mechanical Splices: Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the specified yield strength of the bar.

12A.4.5.6.4 End Bearing Splices:

12A.4.5.6.4.1: In bars required for compression only, the transmission of compressive stress by bearing of square cut ends held in concentric contact by a suitable device is permitted.

12A.4.5.6.4.2: Bar ends shall terminate in flat surfaces within 1-1/2 degrees of a right angle to the axis of the bars and shall be fitted within 3 degrees of full bearing after assembly.

12A.4.5.6.4.3: End bearing splices shall be used only in members containing closed ties, closed stirrups, or spirals.

12A.5 STRENGTH AND DEFORMATION REQUIREMENTS:

12A.5.1 GENERAL: Masonry structures and masonry members shall be designed to have strength at all sections at least equal to the required strength calculated for the factored loads in such combinations as are stipulated in these provisions.

12A.5.2 REQUIRED STRENGTH: The required strength (U) shall be determined in accordance with Chapter 3.

12A.5.3 DESIGN STRENGTH: Design strength provided by a member and its connections to other members, and its cross sections, in terms of flexure, axial load, and shear shall be taken as the nominal strength multiplied by a strength reduction factor (ϕ) as specified in Table 12.5.3.

TABLE 12A.5.3
Strength Reduction Factor (ϕ)

Flexure Without Axial Load	Reinforced masonry	ϕ = 0.85
	Plain masonry	ϕ = 0.60
Axial load and axial load with flexure	Reinforced masonry[a]	ϕ = 0.65
	Reinforced masonry wall frames	$\phi = 0.85 - 2(P_u/A_n f'_m)$, but not less than 0.65
	Plain masonry	ϕ = 0.60
Shear	Reinforced masonry	ϕ = 0.80
	Plain masonry	ϕ = 0.80
Bearing		ϕ = 0.60

[a] The strength reduction factor (ϕ) shall be permitted to increase linearly to 0.85 as the required axial load strength, P_u, decreases from P_n to zero.

12A.5.4 DEFORMATION REQUIREMENTS:

12A.5.4.1: Masonry structures shall be designed to limit drift within the permissible limits set by these provisions.

12A.5.4.2: Deflection calculations for plain masonry members shall be based on uncracked section properties.

12A.5.4.3: Deflection calculations for reinforced masonry members shall be based on a comprehensive analysis or shall be based on the following:

$$\delta_e = \delta_{cr}(1-\alpha) \qquad (12A\text{-}11)$$

where δ_{cr} is the deflection based on cracked section properties and α is based on the following:

$$\alpha = \left[2\left(\frac{M_{cr}}{M_a}\right)^2 - \left(\frac{M_{cr}}{M_a}\right)^3\right]\left(1 - \frac{I_{cr}}{I_n}\right) \qquad (12A\text{-}12)$$

and

$$M_{cr} = Sf_r \qquad (12A\text{-}13)$$

where

M_{cr} = cracking moment strength of the masonry (in.-lb),

M_a = maximum moment in the member at the stage deflection is computed (in.-lb),

I_{cr} = moment of inertia of the cracked section (in.^{4}),

I_n = moment of inertia of the net cross-sectional area of the member (in.^{4}),

S = uncracked section modulus of the wall, and

f_r = modulus of rupture of masonry (psi).

12A.6 FLEXURE AND AXIAL LOADS:

12A.6.1 SCOPE: This section shall apply to the design of masonry members subject to flexure or axial loads or to combined flexure and axial loads.

12A.6.2 DESIGN REQUIREMENTS OF REINFORCED MASONRY MEMBERS:

12A.6.2.1: Strength design of members for flexure and axial loads shall be in accordance with principles of engineering mechanics, satisfaction of applicable conditions of equilibrium, compatibility of strains and in accordance with the following design assumptions:

a. Strain in reinforcement and masonry shall be assumed directly proportional to the distance from the neutral axis, except, for deep flexural members with overall depth to clear span ratio greater than 2/5 for continuous span members and 4/5 for simple span members, where a nonlinear distribution of strain shall be considered.

b. Maximum usable strain (ϵ_{mu}) at the extreme masonry compression fiber shall be assumed equal to 0.003.

c. Stress in reinforcement below the specified yield strength (f_y) shall be taken as the modulus of elasticity (E_s) times the steel strain. For strains greater than those corresponding to the specified yield strength (f_y), the stress in the reinforcement shall be considered independent of strain and equal to the specified yield strength (f_y).

d. Tensile strength of masonry shall be neglected in calculating the flexural strength of a reinforced masonry cross section.

e. Flexural compression in masonry shall be assumed to be an equivalent rectangular stress distribution. Masonry stress of 0.85 times the specified compressive strength (f'_m) shall be assumed to be uniformly distributed over an equivalent compression zone founded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fiber of maximum compressive strain where c is the distance from the neutral axis of the flexural member to the fiber of maximum compressive strain.

12A.6.2.2: Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength (f_y) just as masonry in compression reaches its maximum usable strain of 0.003.

12A.6.2.3: When design axial load strength (ϕP_n) is less than $0.10 f'_m A_n$, the ratio of reinforcement ρ provided shall not exceed 0.35 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.35 factor.

12A.6.2.4: Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The required moment (M_u), shall include the moment induced by relative lateral displacements.

12A.6.3: Design of Plain Masonry Members:

12A.6.3.1: Strength design of members for flexure and axial load shall be in accordance with principles of engineering mechanics and satisfaction of applicable conditions of equilibrium.

12A.6.3.2: Strain in masonry shall be assumed directly proportional to the distance from the neutral axis.

12A.6.3.3: Flexural tension in masonry shall be assumed directly proportional to strain.

12A.6.3.4: Flexural compression in masonry shall be assumed directly proportional to strain for values up to one-third of the compressive strength (f'_m). For values exceeding $1/3f'_m$ the assumptions given in Sec. 12A.6.2.1(b) and (e) shall apply.

12A.6.3.5: Design axial load strength shall be in accordance with Eq. 12A-14 or Eq. 12A-15.

$$\phi P_n = \phi A_n f'_m \left[1 - \left(\frac{h}{(140r)^2} \right) \right] \textit{ for } h/r < 99 \qquad (12A\text{-}14)$$

$$\phi P_n = \phi A_n f'_m \left(\frac{70r}{h} \right)^2 \textit{ for } h/r \geq 99 \qquad (12A\text{-}15)$$

where

ϕ = 0.6, strength reduction factor,

A_n = net cross-sectional area of the masonry (in.^{2}),

f'_m = specified compressive strength of the masonry (psi),

h = effective height of the wall between points of support (in.), and

r = radius of gyration (in.).

12A.7 SHEAR:

12A.7.1 SCOPE: Provisions of this section shall apply for design of members subject to shear.

12A.7.2 SHEAR STRENGTH:

12A.7.2.1: Design of cross sections subjected to shear shall be based on

$$V_u \leq \phi\ V_n(2/3) \qquad (12A\text{-}16)$$

where

V_u = factored shear force at the section considered,

ϕ = 0.80, strength reduction factor, and

V_n = nominal shear strength.

12A.7.2.2: The nominal shear strength shall exceed the shear corresponding to the development of its nominal flexural strength except that the nominal shear strength need not exceed 2.5 times V_u.

12A.7.3 DESIGN OF REINFORCED MASONRY MEMBERS:

12A.7.3.1: Nominal shear strength, V_n, shall be computed as follows:

$$V_n = V_m + V_s \qquad (12A\text{-}17)$$

where

V_m = nominal shear strength provided by masonry in accordance with Sec. 12A.7.3.2 and

V_s = nominal shear strength provided by reinforcement in accordance with Sec. 12A.7.3.3.

For $M/Vd \leq 0.25$:

$$V_{n(max)} = 6\sqrt{f'_m}A_n$$

For $M/Vd \geq 1.00$:

$$V_{n(max)} = 4\sqrt{f'_m}A_n$$

where

A_n = net cross-sectional area of the masonry (in.^{2}) and

f_m' = specified compressive strength of the masonry (psi).

Values of M/Vd between 0.25 and 1.0 may be interpolated.

12A.7.3.2: Nominal shear strength (V_m) provided by masonry for sections not in a plastic hinge zone shall be as follows:

$$V_m = \left[4.0 - 2.0\left(\frac{M}{Vd}\right)\right]\sqrt{f_m'}\,A_n + 0.3P \qquad \text{(12A-18)}$$

where $M/(Vd)$ need not be taken greater than 1.0 and

M = moment on the masonry section due to unfactored design loads (in.-lb),

V = shear on the masonry section due to unfactored loads (psi),

d = depth of member (in.),

A_n = net cross-sectional area of the masonry (in.^{2}),

f_m' = specified compressive strength of the masonry (psi), and

P = axial load on the masonry section due to unfactored design loads (lb).

Nominal shear strength provided by masonry for sections in a plastic hinge zone shall be as follows:

$$V_m = 1.0\sqrt{f_m'}A_n + 0.3P \qquad \text{(12A-19)}$$

12A.7.3.3: Nominal shear strength (V_s) provided by reinforcement shall be as follows:

$$V_s = 0.8\rho_h f_y \qquad \text{(A12-20)}$$

where

ρ_h = ratio of the area of shear reinforcement and

f_y = specified yield strength of the reinforcement.

12A.7.4 DESIGN OF PLAIN MASONRY MEMBERS:

12A.7.4.1: Nominal shear strength (V_n) shall be the lesser of the following:

a. $1.5\sqrt{f'_m} A_n$

b. $120 A_n$

c. $37 A_n + 0.3 N_v$ for running bond masonry not solidly grouted
$37 A_n + 0.3 N_v$ for stack bond masonry with open end units and grouted solid
$60 A_n + 0.3 N_v$ for running bond masonry solidly grouted
$15 A_n$ for stack bond other than open end units grouted

where

f'_m = specified compressive strength of the masonry (psi),

A_n = net cross-sectional area of the masonry (in.^{2}), and

N_v = force acting normal to shear surface (lb).

12A.8 SPECIAL REQUIREMENTS FOR MASONRY BEAMS:

12A.8.1: The spacing of lateral supports shall be determined by the required strength of out-of-plane loading but it shall not be greater than the following:

a. Spacing between lateral supports shall not exceed 50 times the least width of beams with horizontal reinforcement of a minimum of two bars and with confinement reinforcement.

b. The spacing between lateral supports shall not exceed 32 times the least width of the beam of beams which do not comply with the minimum number of bars of confinement reinforcement.

12A.8.2: Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

12A.8.3: At any section of a flexural member, except as provided below where positive reinforcement is required by analysis, the ratio ρ shall not be less than that given by:

$$\rho \geq \frac{120}{f_y} \qquad (12A\text{-}21)$$

where f_y is the specified yield strength of the reinforcement.

In masonry beams, where a concrete floor provides a flange and where the web is in tension, the ratio ρ shall be computed for this purpose using width of web. Alternatively, area of reinforcement provided at every section, positive or negative, shall be at least one-third greater than that required by analysis.

12A.8.4 DEEP FLEXURAL MEMBERS:

12A.8.4.1: Flexural members with overall depth to clear span ratios greater than 2/5 for continuous spans, or 4/5 for simple spans, shall be designed as deep flexural members taking into account nonlinear distribution of strain and lateral buckling.

12A.8.4.2: Minimum flexural tension reinforcement shall conform to Sec. 12A.8.3.

12A.8.4.3: Minimum horizontal and vertical reinforcement ratio in the side faces of deep flexural members shall be 0.001.

12A.9 SPECIAL REQUIREMENTS FOR COLUMNS:

12A.9.1: Area of longitudinal reinforcement for columns shall be not less than 0.005 nor more than 0.04 times cross-sectional area of the column.

12A.9.2: Minimum number of longitudinal bars in columns shall be 4.

12A.9.3: Lateral ties shall be provided to resist shear and shall comply with the following:

a. Lateral ties shall be at least 1/4 inch in diameter.

b. Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 lateral tie diameters, nor the least cross sectional dimension of the column.

c. Lateral ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135 degrees and no bar shall be farther than 6 inches clear on each side along the lateral tie from such a laterally supported bar. Lateral ties shall be placed in either a mortar joint or grout. Where longitudinal bars are located around the perimeter of a circle, a complete circular lateral tie is permitted. Lap length for circular ties shall be 84 tie diameters.

d. Lateral ties shall be located vertically not more than one-half lateral tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than one-half a lateral tie spacing below the lowest horizontal reinforcement in beam, girder, slab, or drop panel above.

e. Where beams or brackets frame into a column from four directions, lateral ties may be terminated not more than 3 inches below lowest reinforcement in shallowest of such beams or brackets.

12A.9.4: The reinforcement corresponding to balanced strain conditions with flexure only shall be calculated. The moments in the member due to rotational restraint and interstory drift shall be calculated. The maximum reinforcement provided in the member for combined axial and flexure shall be 35 percent of the balanced reinforcement calculated for flexural only.

12A.10 SPECIAL REQUIREMENTS FOR WALLS:

12A.10.1: When the wall limit state is out of plane flexure, the nominal flexural strength of the wall shall be at least equal to 1.3 times the cracking moment strength of the wall.

12A.11 SPECIAL REQUIREMENTS FOR SHEAR WALLS:

12A.11.1: REINFORCEMENT: Maximum spacing of horizontal reinforcement shall not exceed six times nominal wall thickness or 48 inches, whichever is less.

12A.11.2 CONFINEMENT OF COMPRESSIVE STRESS ZONE:

12A.11.2.1: Confinement ties shall be provided in accordance with this section when the corresponding masonry compressive strain exceeds 0.00015.

12A.11.2.2: The minimum horizontal length of the confinement region shall be three times the thickness of the wall.

12A.11.2.3: Confinement ties shall comply with the requirements of Sec. 12A.9.3, shall have a closed perimeter and terminated by a standard 135 or 180 degree hook, and shall consist of a minimum of No. 3 bars at a maximum 8-inch spacing or equivalent.

12A.11.3 FLANGED SHEAR WALLS:

12A.11.3.1: Wall intersections shall be considered effective in transferring shear when the following requirements are met:

a. The face shells of the masonry units shall be removed and the intersection shall be fully grouted.

b. All horizontal reinforcement shall be continuous through the intersection.

12A.11.3.2: Flange shall be considered effective in resisting applied loads.

12A.11.3.3: The width of flange considered effective in compression on each side of the web shall be taken equal to 1/6 of the wall height or shall be equal to the actual flange on either side of the web wall, whichever is less.

12A.11.3.4: The width of flange considered effective in tension on each side of the web shall be taken equal to 1/3 of the wall height or shall be equal to the actual flange on either side of the web wall, whichever is less.

12A.11.4 COUPLED SHEAR WALLS:

12A.11.4.1 Design of Coupled Shear Walls: Structural members which provide coupling between shear walls shall be designed to reach their moment or shear resistance limit state before either shear wall reaches its moment or shear limit state. Analysis of coupled shear walls shall conform to accepted principles of mechanics.

12A.11.4.2 Shear Strength of Coupling Beams: The nominal shear strength (V_m) of the coupling beams shall exceed the shear demand calculated:

$$V_n \geq \frac{M_1 + M_2}{L} \tag{12A-22}$$

where

V_n	=	shear demand,
M_1 and M_2	=	nominal moment strength at the ends of the beam, and
L	=	length of the beam between the shear walls.

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor system. The effective width of the reinforced concrete system shall be six times the floor or roof slab thickness.

12A.12 WALL FRAMES:

12A.12.1: These requirements have been developed for masonry moment resisting wall frame structures for which the design forces related to earthquake motion have been determined on the basis of energy dissipation in the inelastic range of response. They may also be used for frame members subject to gravity and wind loading.

12A.12.2: The calculation of required strength of the members shall be in accordance with engineering mechanics and shall consider the effects of the relative stiffness degradation of the beams and columns. The frame analysis may be an iterative process in which the stiffness of the members are reduced to their effective stiffness.

The limit state of flexural yielding of members is limited to the flexural beams at the face of the columns and to the bottom of the columns at the base of the building.

An additional limit state is the interstory drift or the overall drift limit of the top of the wall frame. The drift ratios for interstory drift shall conform to Sec. 4.6.1. The stiffness used for the wall frame shall be that used for calculation of the effective period for calculation of lateral loading.

12A.12.3: The maximum useable strain (ϵ_{mu}) at the extreme masonry compression fiber shall not exceed 0.0025, unless special confinement is utilized.

12A.12.4: The development of nonlinear hinges in a wall frame shall be limited to the beams and the bottom of the column at the base of the wall frame.

12A.12.5 REINFORCEMENT:

12A.12.5.1: The nominal moment strength at any section along a member shall not be less than 1/2 of the higher moment strength provided at the two ends of the member.

12A.12.5.2: Lap splices are permitted only within the center half of the member length.

12A.12.5.3: Welded splices and mechanical connections may be used for splicing the reinforcement at any section, provided not more than alternate longitudinal bars are spliced at a section, and the distance between splices on alternate bars is at least 24 inches along the longitudinal axis.

12A.12.5.4: Reinforcement shall have a specified yield strength of 60 ksi.

12A.12.6 BEAMS IN WALL FRAMES:

12A.12.6.1: Factored axial compression force on the beam shall not exceed 0.10 times the net cross-sectional area of the beam (A_n) times the specified compressive strength (f'_m).

12A.12.6.2: Beams interconnecting lateral load resisting elements shall be limited to a reinforcement ratio of 35 percent of balanced design reinforcement. All reinforcement in the beam and adjacent to the beam in a reinforced concrete roof or floor system shall be used to calculate the reinforcement ratio. The reinforcement in the body of the beam shall be uniformly distributed throughout the depth of the beam.

12A.12.6.3: Clear span for the beam shall not be less than 4 times its depth.

12A.12.6.4: Nominal depth of the beam shall not be less than 4 units or 32 inches, whichever is greater. The nominal depth to nominal width ratio shall not exceed 4.

12A.12.6.5: Nominal width of the beam shall not be less than 8 inches or that required by Sec. 12A.8.1 or 1/26 of the clear span between column faces, whichever is less.

12A.12.6.6 Longitudinal Reinforcement:

12A.12.6.6.1: Longitudinal reinforcement shall not be spaced more than 8 inches on center.

12A.12.6.6.2: Longitudinal reinforcement shall be uniformly distributed along the depth of the beam.

12A.12.6.6.3: Minimum reinforcement ratio shall be $130/f_y$ where f_y is the specified yield strength of the reinforcement.

12A.12.6.7. Transverse Reinforcement:

12A.12.6.7.1: Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180-degree hook.

12A.12.6.7.2: Within an end region extending one beam depth from space frame column faces and at any region at which beam plastic hinges may form during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one-fourth the nominal depth of the beam.

12A.12.6.7.3: The maximum spacing of transverse reinforcement shall not exceed 1/4 the nominal depth of the beam or that required for shear strength.

12A.12.6.7.4: Minimum transverse reinforcement ratio shall be 0.0015.

12A.12.7 COLUMNS IN WALL FRAMES:

12A.12.7.1: Factored axial compression force on the wall frame column shall not exceed 0.30 times the net cross-sectional area of the beam (A_n) times the specified compressive strength (f'_m) or 60 percent of the balanced design axial load.

12A.12.7.2: Nominal dimension of the column parallel to the plane of the wall shall not be less than two full units or 24 inches, whichever is greater.

12A.12.7.3: Nominal dimension of the column shall be perpendicular to the plane of the wall frame not be less than 8 inches or 1/14 of the clear height between beam faces, whichever is greater.

12A.12.7.4 Longitudinal Reinforcement:

12A.12.7.4.1: A minimum of 4 longitudinal bars shall be provided at all sections of every wall frame column member.

12A.12.7.4.2: The flexural reinforcement shall be essentially uniformly distributed across the member depth.

12A.12.7.4.3: The nominal moment strength at any section along a member shall be not less than 1.8 times the cracking moment strength and the minimum reinforcement ratio shall be $130/f_y$ where f_y is the specified yield strength of the reinforcement.

12A.12.7.5 Transverse Reinforcement:

12A.12.7.5.1: Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180 degree hook.

12A.12.7.5.2: The maximum spacing of transverse reinforcement shall not exceed 1/4 the nominal dimension of the column parallel to the plane of the wall frame.

12A.12.7.5.3: Minimum transverse reinforcement ratio shall be 0.0015.

12A.12.8 WALL FRAME BEAM-COLUMN INTERSECTION:

12A.12.8.1: Beam-column intersections dimensions in masonry wall frames shall be proportioned such that the wall frame column depth in the plane of the frame exceeds 60 times the diameter of beam longitudinal reinforcement passing through the beam-column intersection.

Beam depth shall exceed 40 times the diameter of the wall frame column longitudinal reinforcement passing through the beam-column intersection.

Nominal shear strength of beam-column intersections shall exceed the shear occurring when wall frame beams develop their nominal flexural strength.

12A.12.8.2: Beam longitudinal reinforcement terminating in a wall frame column shall be extended to the far face of the column and anchored by a standard 90° or 180° hook bent back into the wall frame column.

Special horizontal shear reinforcement crossing a potential diagonal beam column shear crack shall be provided such that:

$$A_s \geq \frac{0.5\,V_n}{f_y} \tag{12A-23}$$

where

A_s = cross-sectional area of reinforcement (in.^{2}),

V_n = nominal shear strength (lb) and

f_y = specified yield strength (psi).

Special horizontal shear reinforcement shall be anchored by a standard hook around the extreme wall frame column reinforcing bars.

12A.12.8.3 Shear Strength: The nominal horizontal shear stress at the beam-column intersection shall not exceed 350 psi.

Appendix A

DIFFERENCES BETWEEN THE 1988 AND 1991 EDITIONS OF THE NEHRP RECOMMENDED PROVISIONS

INTRODUCTION

The 1991 Edition of the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* reflects a number of substantive and editorial/format changes. The section numbers cited below pertain to the 1991 Edition.

EDITORIAL AND FORMAT CHANGES

The editorial and format changes made for the 1991 Edition of the *NEHRP Recommended Provisions* are not highlighted in the text. Generally, these changes clarify the text, present requirements in a better organized manner, explicitly identify exceptions to requirements, incorporate the definition of a symbol whenever it appears in the text, and assign a table the number of the section to which it pertains. The Provisions Update Committee deemed these changes to be nonsubstantive and, therefore, not in need of balloting by the BSSC member organizations.

CHANGES IN CHAPTER 1, GENERAL PROVISIONS

The first paragraph of Sec. 1.2, Scope, has been modified to clarify that the *Provisions* apply to existing buildings only when a change of Seismic Hazard Exposure Group occurs as covered in Sec. 1.3.3, Change of Use. Exception 1 under Sec. 1.2 has been modified to clarify that the *Provisions* document does not apply to detached one- and two-family dwellings in areas where the effective peak velocity-related acceleration (A_v) value is less than 0.15.

The phrase "including tenant improvements" has been added to the last sentence of Sec. 1.3.1, New Buildings, to ensure that separately designed and performed nonstructural construction meets the requirements of the *Provisions*. The exception to Sec. 1.3.1 has been modified to clarify that the *Provisions* document applies to attached single-family dwellings since some codes consider multiple attached dwellings to be single-family dwellings.

Sec. 1.3.2, Additions to Existing Buildings, has been revised to clarify the intent. The changes were approved by the required two-thirds majority of BSSC members during balloting; however, several members raised issues that remain in need of attention during the next updating of the *Provisions*. These members maintain that Item 3 of Subsection 1.3.2.2 should address the entire structure rather than individual elements.

An exception has been added to Sec. 1.3.3, Change of Use, because it was deemed to be unreasonable to require that every building undergoing a change of Seismic Hazard Exposure Group comply with the provisions. This exception is considered to be a reasonable alternative in that retrofitting is required for all such changes in higher seismic risk areas and for those buildings being converted to Seismic Hazard Exposure Group III in lower seismic risk areas.

A footnote has been added at Sec. 1.4.1, Seismic Ground Acceleration Maps, referring to the new maps and approach for establishing design ground motions presented in the Appendix to Chapter 1 and emphasizing the preliminary nature of this appendix. A sentence also has been added to Sec. 1.4.1 indicating that site-specific ground motions, when used or required, should be developed on the same basis as Maps 1 and 2 for effective peak acceleration (A_a) and effective peak velocity-related acceleration (A_v)--that is, with a 90 percent probability of nonexceeedance in 50 years.

A footnote has been added to Table 1.4.1.1, Coefficients A_a and A_v, to clarify that the values of A_a and A_v used in equations should not be less than 0.05 even though some provisions refer to "areas with a value of A_v less than 0.05." The intent of Map Area 1 in Maps 1 and 2 is that the maximum value of A_a and A_v is less than 0.05; however, the footnote proposed here is necessary since it is the intent of these provisions that the value be 0.05 when these terms are used in an equation.

Section 1.4.2.4, Multiple Use, has been modified to indicate that a building should be assigned to the highest Seismic Hazard Exposure Group occupying any portion of the building.

Section 1.4.2.5, Group III Building Protected Access, has been modified to clarify that the protected access provisions also apply to all buildings on the same property since it is the intent of the provisions that they address the situation where a Group III building must be accessed by passing near another building on the same property.

Section 1.4.2.6, Group III Function, has been modified to draw attention to the vulnerability of utility systems especially in certain site conditions.

Sections 1.6.2.2, 1.6.2.4, and 1.6.2.5 have been revised to state how often special inspection is to be performed. These changes were approved by the required two-thirds majority of BSSC members during balloting; however, several members raised issues that remain in need of attention during the next updating of the *Provisions*. Those concerned believe that in Sec. 1.6.2.2.1 for reinforced concrete special moment frames, inspection should be essentially continuous during the placing of reinforcing.

Section 1.6.2.6.2 has been revised to indicate that special inspection is to be performed for slip-critical and full-pretensioned high-strength bolts only.

Section 1.6.3.1.1 has been revised to require only the data for ASTM 615 reinforcing steel that is actually generated by certified mill test reports.

The 1991 Edition's Appendix to Chapter 1, Preliminary Spectral Response Maps and Method, is completely different from the appendix that appeared in the 1988 Edition. The 1991 version introduces new maps (Maps 5 though 12) defining seismic ground motion in terms of spectral response accelerations at 0.3 second and at 1.0 second. The text of the appendix shows changes that need to be made in the *Provisions* to use the new maps. This change is in keeping with the trend towards use of spectral values in the development of design ground motions. Note that the appendix title stresses the preliminary nature of the material and that the appendix preface emphasizes the fact that the appendix is included to obtain broad professional review and feedback. The *Commentary* for the Appendix to Chapter 1 has been revised to reflect the new appendix text and maps.

The *Commentary* for Sec. 1.4.1 has been revised to reflect changes in the *Provisions*, to expand on current understanding of the earthquake behavior of Soil Profile Types S_3 and S_4, and to correct some plotting errors in the figures.

CHANGES IN CHAPTER 2, DEFINITIONS AND SYMBOLS

A number of editorial changes have been made in Chapter 2 including the listing of all symbols used in the materials chapters (Chapters 9 through 12). The substantive changes include a modification of the definitions for light framed walls, live load, and snow load. In addition, the definitions of C_T and P_u have been modified to be consistent with the remainder of the text.

CHANGES IN CHAPTER 3, STRUCTURAL DESIGN REQUIREMENTS

Table 3.2, Site Coefficient, presents revised definitions of Soil Profile Types S_3 and S_4 that are consistent with those in the *Uniform Building Code*.

Section 3.5.3, Seismic Performance Categories D and E, has been revised to remind the analyst that the provisions of Chapter 4, Equivalent Lateral Force Procedure, may not be adequate in all cases (e.g., the equivalent static analysis of Chapter 4 is inappropriate for buildings that have only vertical irregularities of Type 1, 2, or 3 in Table 3.4.2 and that have a height exceeding five stories or 65 feet and for all buildings exceeding 240 feet in height). This modification is something of a stop-gap measure since it is hoped that specific requirements concerning the circumstances that require special consideration of dynamic response characteristics for these low-rise structures can be developed following an extensive series of dynamic analyses of irregular buildings. Note that site-specific response spectra are required under some circumstances for analysis of Seismic Performance Category D and E buildings sited near faults and on Type S_4 soils. The *Commentary* for Sec. 3.5 also has been revised to provide guidance concerning the performance of three-dimensional response spectrum analysis.

A new requirement, Sec. 3.6.3, Seismic Performance Category C, has been added to assure a more accurate structural analysis for buildings in which the vertical portions of the structural seismic force resisting system are not parallel to the principal axes, because the conventional approximation is less accurate for this class of buildings.

Section 3.7, Combination of Load Effects, has been revised for two reasons: to include the amplified load factor required by introduction of load and resistance factor design (LRFD) requirements into Chapter 10, Steel, and to bring the load requirements into closer agreement with ASCE 7.

A paragraph highlighting the importance of redundancy has been added to the *Commentary* as Sec. 3.7.4, Nonredundant Systems.

CHANGES IN CHAPTER 4, EQUIVALENT LATERAL FORCE PROCEDURE

Section 4.2 has been modified to reflect changes concerning snow load modification in Sec. 3.7, Combination of Load Effects.

Section 4.2.1 has been modified to eliminate the reduction factor for soil profile types S_3 and S_4 based on recent experience concerning amplification of soft clay sites.

Sections 4.2.2.1, Approximate Fundamental Period for Concrete and Steel Moment Resisting Frame Buildings, and 4.2.2.2, Approximate Fundamental Period for All Other Buildings, have been revised to eliminate the older period formula used for buildings other than steel and concrete frames. The older formula has been found to be unrealistic in many circumstances, particularly for those structures with interior braced frames or shear walls.

Section 4.6.2, *P*-Delta Effects, has been revised to require that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$, where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 9 through 12 of the *Provisions*. The *Commentary* for Sec. 4.6 has been revised to discuss this new requirement and the references have been updated.

CHANGES IN CHAPTER 5, MODAL ANALYSIS PROCEDURE

Section 5.5 has been revised to eliminate the reduction factor for soil profile types S_3 and S_4 based on recent experience concerning amplification of soft clay sites.

A new exception has been added to Sec. 5.5 and Exception 2 has been modified to remind the analyst that the provisions of Chapter 5 may not be adequate for buildings with certain vertical irregularities.

Section 5.8, Design Values, has been modified to permit the complete quadratic combination (CQC) technique to be used to combine modal values.

CHANGES IN CHAPTER 6, SOIL-STRUCTURE INTERACTION

The *Commentary* for Sec. 6A.2.2 and 6A.2.3 has been modified to note that it may be unconservative to use inelastic stiffness in determining theoretical *P*-delta response.

CHANGES IN CHAPTER 7, FOUNDATION DESIGN REQUIREMENTS

Section 7.5.1, Investigation, has been revised to indicate that investigations for the more common geotechnical hazards such as slope instability and liquefaction should be performed for Seismic Performance Category D and E buildings. *Commentary* Sec. 7.5.1 also has been revised and expanded to reflect current information on geotechnical hazards and the references are updated.

Commentary Sec. 7.4.2, Pole-Type Structures, has been revised to indicate that soil conditions suitable for pole-type structures where there is no significant earthquake risk may not be suitable where there is a likelihood of strong motion.

CHANGES IN CHAPTER 8, ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS AND SYSTEMS

Storage racks have been added to Table 8.2.2, Architectural Component Seismic Coefficient (C_c) and Performance Criteria Factor (P).

A new provision, Sec. 8.2.6, Ceilings, has been added to require consideration of the interaction of the ceiling system and architectural, mechanical, and electrical systems or components with one another and their effect on the total performance of the ceiling system. Placement of light fixtures or other components without appropriately located bracing may have a direct effect on the performance of the ceiling under seismic action. This new requirement is discussed in a new *Commentary* section.

The term "code required" has been deleted from the first two line items (fire protection equipment and systems and emergency or standby electrical systems) of Table 8.3.2a, Mechanical and Electrical Component and System Seismic Coefficient (C_c) and Performance Criteria Factor (P), because these systems should satisfy seismic performance requirements whether they are code required or not.

Section 8.3.4, Component Attachment, has been modified to emphasize the importance of displacement for heavy pieces of mechanical equipment. A corresponding paragraph has been added to the *Commentary* for Chapter 8.

A new requirement, Sec. 8.3.7, Site-Specific Considerations, has been added to draw attention to possible site specifics that may interrupt critical utility functions; building systems that are required to remain functional should be designed with these concerns in mind.

A new subsection discussing tenant improvements has been added to the "Related Concerns" section of the *Commentary* for Chapter 8.

CHANGES IN CHAPTER 9, WOOD

Chapter 9, Wood, has been revised to correct terminology and add references to bring the chapter into conformance with latest industry usage, to modify conventional construction criteria, and to modify bracing requirements applicable to conventional construction in Seismic Performance Categories C and D. The *Commentary* also has been revised.

The revised Chapter 9 was approved by the required two-thirds majority of the BSSC members during balloting. However, several members raised issues (concerning, for example, the requirements for braced wall panels for conventional construction, clarity of presentation of the requirements, and the need for cross-referencing within the chapter) that will be addressed during the next updating of the *Provisions*.

CHANGES IN CHAPTER 10, STEEL

Chapter 10 has been revised to reflect the new American Institute of Steel Construction (AISC) load and resistance factor design (LRFD) seismic appendix. The text of the AISC LRFD seismic appendix has been incorporated into the body of Chapter 10 (rather than adopting the appendix by reference) because the appendix uses ANSI A58.1 as a load model. The final section of the revised chapter (Sec. 10.14) presents an alternative to LRFD that permits use of allowable stress design procedures as modified in that section. The references have been updated and the *Commentary* for Chapter 10 has been revised to reflect the new provisions.

CHANGES IN CHAPTER 11, REINFORCED CONCRETE

Section 11.1.1.6 has been revised to be consistent with the *Uniform Building Code.*

A new requirement, Sec. 11.1.1.9, has been added to present requirements for coupling beams and a section on this new requirement has been added to the *Commentary* for Chapter 11.

Section 11.1.1.15 has been deleted because it is no longer needed in view of a revision to Sec. 11.9.3.

Section 11.2, Strength of Members and Connections, has been revised to present provisions based on the preferred strength design method.

The title of Sec. 11.7.3 has been changed to be consistent with the revised section of ACI 318-89 to which it refers and the text has been changed to eliminate a conflict between Sec. 3.3.4.3 of the *Provisions* and Sec. 21.8.1 of ACI 318-89.

CHANGES IN CHAPTER 12, MASONRY

A new appendix has been added to Chapter 12, Masonry, to present an alternative seismic design procedure for masonry based on limit states design criteria. As noted in the preface of the appendix, the information is included for information and it would be premature to base code provisions on the appendix; however, user review, trial design, and comment are encouraged. A discussion of the new appendix has been added to the *Commentary* for Chapter 12.

Appendix B

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