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1994 Edition

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS

Part 1 - Provisions

Issued by FEMA in furtherance of the Decade for Natural Disaster Reduction

BUILDING SEISMIC SAFETY COUNCIL

Program on Improved Seismic Safety Provisions

A council of the National Institute of Building Sciences

1994 Edition

NEHRP RECOMMENDED PROVISIONS FOR 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.

See Appendix E of the *Commentary* volume for a full description of the BSSC program.

BSSC Program on Improved Seismic Safety Provisions

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

1994 EDITION

Part 1: PROVISIONS

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

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
1994

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:

NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, 2 volumes and maps, 1994

NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1991 Edition, 2 volumes and maps, 1991

Guide to Use of the 1991 NEHRP Recommended Provisions in Earthquake-Resistant Design of Buildings, 1995

Non-Technical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1995

Seismic Considerations for Communities at Risk, 1995

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, 1995

Seismic Considerations: Office Buildings, 1995

Societal Implications: Selected Readings, 1986.

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

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.

PREFACE

One of the primary goals of the Federal Emergency Management Agency (FEMA) and the National Earthquake Hazards Reduction Program (NEHRP) is to reduce, or mitigate, the nation's losses that result from the earthquake hazard. In order to accomplish this goal, FEMA is committed to encouraging design and building practices that effectively address the earthquake hazard and minimize the damage that can result. To help encourage these practices, FEMA has had a long history of working with the design and engineering communities and the model code and standards writing organizations. One of the most effective of these relationships has been with the Building Seismic Safety Council (BSSC).

The 1994 Edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* is the third update of this consensus document and marks yet another milestone in the continuing effort to provide a resource that can be utilized by the nation's designers, engineers, model code organizations, and regulatory bodies.

Looking ahead, FEMA has contracted with the BSSC to begin the next update cycle that will result in the 1997 Edition of the *Provisions*. In addition to the regular update process, this undertaking provides for the development of a design procedure based on spectral response seismic hazard maps presently being revised by the U.S. Geological Survey.

FEMA wishes to express its sincere thanks to the large number of volunteer experts and the BSSC Board of Direction and staff who gave freely of their time and talents. Without the dedication and hard work of these individuals, this document and all it represents would not have been possible.

Federal Emergency Management Agency

INTRODUCTION and ACKNOWLEDGMENTS

The 1994 Edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings** is the fourth edition of the document and, like the 1985, 1988 and 1991 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, Appendix E of the *Commentary* volume presents a detailed description of the effort.)

During the effort to update the 1991 Edition for issuance as the 1994 Edition, the BSSC Provisions Update Committee (PUC), its 12 technical subcommittees and Design Values Panel, 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 1991 Edition was published and considered new experience and research data that had become available. Their work resulted in proposals for change to the 1991 Edition that were balloted by the Council membership in April-May 1994. 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 September 1994. This 1994 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 1985 Edition of the *Provisions* was based. A summary of the differences between the 1991 and 1994 Editions of the *Provisions* is presented in Appendix A of this volume.

In presenting this 1994 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 three editions of the *Provisions*, the 1994 update effort, and the information development/dissemination activities conducted to stimulate use of the *Provisions* has benefited 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 (Appendix B of this volume presents a listing of 1994 update process participants). Nevertheless, the 1994 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;

* Note that earlier editions of the *Provisions* were entitled *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*.

- The members of the 12 PUC Technical Subcommittees and Design Values Panel; and

- Michael Mahoney, FEMA Physical Scientist, who served as the update effort project officer.

Appreciation also is due to the BSSC Executive Director James R. Smith and BSSC staff members O. Allen Israelsen, Claret Heider and Karen Smith, all of whose talents and experience were crucial to conduct of the program. Thanks also are due to Richard McConnell, who served as the BSSC's metric consultant.

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 1994 update effort or who otherwise participated in the BSSC program that resulted in the *NEHRP Recommended Provisions*.

James Beavers
Chairman, BSSC Board of Direction

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NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS

1994 EDITION

Part 1: PROVISIONS

NOTE

The structure of the *Provisions* has been revised for the 1994 Edition:

- The 1991 Chapter 2 definitions and symbols have been moved to "Glossary" and "Notations" sections at the conclusion of the *Provisions* volume.
- 1991 Chapter 3, "Structural Design Requirements"; Chapter 4, "Equivalent Lateral Force Procedure"; Chapter 5, "Modal Analysis Procedure"; and Chapter 6, "Soil-Structure Interaction Effects" have been combined into one chapter for the 1994 Edition -- Chapter 2, "Structural Design Criteria, Analysis, and Procedures." New sections with provisions concerning nonbuilding structures and seismically isolated structures have been incorporated into this chapter as has an appendix on passive energy dissipation systems.
- 1991 Chapter 7 ("Foundation Design Requirements") has become 1994 Chapter 4.
- 1991 Chapter 8 ("Architectural, Mechanical, and Electrical Components and Systems") has become 1994 Chapter 3.
- 1991 Chapter 9 ("Wood") remains Chapter 9 in the 1994 Edition.
- 1991 Chapter 10 ("Steel"), has become 1994 Chapter 5.
- 1994 Chapter 11 ("Reinforced Concrete") has become 1994 Chapter 6 and a new appendix on precast concrete elements interconnected using dry connections has been added.
- 1991 Chapter 12 ("Masonry") has become 1994 Chapter 8. Note that the 1991 "Appendix to Chapter 8" has become the main body of the chapter and the 1991 chapter text has been moved to an appendix for the 1994 Edition.
- A new Chapter 7, "Composite Steel and Concrete Structure Design Requirements," has been added for the 1994 Edition.
- Figures and equations have been relabeled in the 1994 Edition with the section numbers in which they appear (tables were so numbered beginning in the 1991 Edition).

To facilitate use of the 1994 Edition by those familiar with the 1991 Edition section numbers, Appendix A of this *Provisions* volume concludes with a comparison of the section numbers, etc., of the two editions.

Those portions of the 1994 Edition of the *Provisions* that are substantively different from the 1991 Edition are identified in the margins as follows:

Additions
or
revisions

Deletions

Not highlighted are editorial and format changes; however, Appendix A of the this volume presents a summary of the substantive differences between the 1991 and 1994 Editions and includes an explanation of the format changes and a comparison of 1991 and 1994 chapter and section numbers.

Chapter 1

GENERAL PROVISIONS

1.1 PURPOSE: These provisions present criteria for the design and construction of buildings and nonbuilding structures subject to earthquake ground motions. Their purposes are to minimize the hazard to life for all buildings and nonbuilding structures, 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. Because of the complexity of and the great number of variables involved in seismic design (e.g., the variability in ground motion, soil types, dynamic characteristics of the structure, material strength properties, quality assurance and control, and construction practices), these provisions present only minimum criteria in general terms. These minimum criteria are 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. It must be emphasized that absolute safety and prevention of damage even in an earthquake event with a reasonable probability of occurrence cannot be achieved economically for most buildings.

The "design earthquake" ground motion levels specified herein may result in both structural and nonstructural damage. For most structures designed and constructed according to these provisions, it is expected that structural damage from a major earthquake may be repairable but it may not be economical. This would depend upon a number of factors including the structure framing type, materials, and details of construction actually used. 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 nonbuilding structure, and portion thereof, shall be designed and constructed to resist the effects of earthquake motions as prescribed by these provisions. Additions to existing buildings and nonbuilding structures also shall be designed and constructed to resist the effects of earthquake motions determined as prescribed by these provisions. Existing buildings and nonbuilding structures and alterations and repairs to existing buildings need only comply with these provisions when required by Sec. 1.3.2 and 1.3.3.

EXCEPTIONS:

1. Detached one- and two-family dwellings that are located at sites where the seismic coefficient C_a is less than 0.15 are exempt from all requirements of these provisions.

2. Detached one- and two-family wood frame dwellings with not more than 2 stories that are located at sites where the seismic coefficient C_a is equal to or greater than 0.15 and that satisfy the limitations and requirements of Sec. 9.10 are exempt from all other requirements of these provisions.

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

4. 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. 2.2.5.1.

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 2 and 4. Materials used in construction and components made of these materials shall be designed and constructed to meet the requirements of Chapters 5 through 9. Architectural, electrical, and mechanical systems and components including tenant or owner improvements shall be designed in accordance with Chapter 3.

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.

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.

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 should be used.

1.4.2 SEISMIC COEFFICIENTS: The values of seismic coefficients (C_a and C_v) shall be determined from Sec. 1.4.2.3 or Tables 1.4.2.4a and 1.4.2.4b based on Soil Profile Types defined as follows:

A Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/sec (1500 m/s)

B Rock with 2,500 ft/sec $< \bar{v}_s \leq 5{,}000$ ft/sec (760 m/s $< \bar{v}_s \leq 1500$ m/s)

C Very dense soil and soft rock with 1,200 ft/sec $< \bar{v}_s \leq 2{,}500$ ft/sec (360 m/s $< \bar{v}_s \leq 760$ m/s) or with either $\bar{N} > 50$ or $\bar{s}_u \geq 2{,}000$ psf (100 kPa)

D Stiff soil with 600 ft/sec $\leq \bar{v}_s \leq 1{,}200$ ft/sec (180 m/s $\leq \bar{v}_s \leq 360$ m/s) or with either $15 \leq \bar{N} \leq 50$ or 1,000 psf $\leq \bar{s}_u \leq 2{,}000$ psf (50 kPa $\leq \bar{s}_u \leq 100$ kPa)

E A soil profile with $\bar{v}_s < 600$ ft/sec (180 m/s) or any profile with more than 10 ft (3 m) of soft clay defined as soil with PI > 20, $w \geq 40$ percent, and $s_u < 500$ psf (25 kPa)

F Soils requiring site-specific evaluations:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.

2. Peats and/or highly organic clays ($H > 10$ ft [3 m] of peat and/or highly organic clay where H = thickness of soil)

3. Very high plasticity clays ($H > 25$ ft [8 m] with PI > 75)

4. Very thick soft/medium stiff clays ($H > 120$ ft [36 m])

EXCEPTION: When the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type D shall be used. Soil Profile Types E or F need not be assumed unless the regulatory agency determines that Types E or F may be present at the site or in the event that Types E or F are established by geotechnical data.

1.4.2.1 Steps for Classifying a Site (also see Table 1.4.2.1 below):

Step 1: Check for the four categories of Soil Profile Type F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Soil Profile Type F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 10 ft (3 m) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), $w \geq 40$ percent, and PI > 20. If these criteria are satisfied, classify the site as Soil Profile Type E.

Step 3: Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified by the definitions in Sec. 1.4.2.2:

a. $\bar{v}_s$ for the top 100 ft (30 m) ($\bar{v}_s$ method)

b. $\bar{N}$ for the top 100 ft (30 m) ($\bar{N}$ method)

c. $\bar{N}_{ch}$ for cohesionless soil layers (PI < 20) in the top 100 ft (30 m) and average $\bar{s}_u$ for cohesive soil layers (PI > 20) in the top 100 ft (30 m) ($\bar{s}_u$ method)

TABLE 1.4.2.1
Soil Profile Type Classification

Soil Profile Type	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	< 600 fps (< 180 m/s)	< 15	< 1,000 psf (< 50 kPa)
D	600 to 1,200 fps (180 to 360 m/s)	15 to 50	1,000 to 2,000 psf (50 to 100 kPa)
C	1,200 to 2,500 fps (360 to 760 m/s)	> 50	> 2,000 (> 100 kPa)

NOTE: If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Soil Profile Type E instead of D).

The shear wave velocity for rock, Soil Profile Type B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type C.

The hard rock, Soil Profile Type A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 ft (30 m), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_{s.}$

The rock categories, Soil Profile Types A and B, shall not be used if there is more than 10 ft (3 m) of soil between the rock surface and the bottom of the spread footing or mat foundation.

1.4.2.2 Definitions: The definitions presented below apply to the upper 100 ft (30 m) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there are a total of *n* distinct layers in the upper 100 ft (30 m). The symbol *i* then refers to any one of the layers between 1 and *n*.

v_{si} is the shear wave velocity in ft/sec (m/s).

d_i is the thickness of any layer between 0 and 100 ft (30 m).

$\bar{v}_s$ is:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad (1.4.2\text{-}1)$$

where $\sum_{i=1}^{n} d_i$ is equal to 100 ft (30 m)

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/ft as directly measured in the field without corrections.

$\bar{N}$ is:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad (1.4.2\text{-}2)$$

$\overline{N}_{ch}$ is:

$$\overline{N_{ch}} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad (1.4.2\text{-}3)$$

where $\sum_{i=1}^{m} d_i = d_s$.

(Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m).

s_{ui} is the undrained shear strength in psf (kPa), not to exceed 5,000 psf (250 kPa), ASTM D2166-91 or D2850-87.

$\overline{s}_u$ is

$$\overline{s_u} = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad (1.4.2\text{-}4)$$

where $\sum_{i=1}^{k} d_i = d_c$.

d_c is the total thickness (100 - d_s) of cohesive soil layers in the top 100 ft (30 m).

PI is the plasticity index, ASTM D4318-93.

w is the moisture content in percent, ASTM D2216-92.

1.4.2.3 Site Coefficients: The values for site coefficients F_a and F_v are as indicated in Tables 1.4.2.3a and 1.4.2.3b, respectively, and are used to determine seismic coefficients C_a and C_v as follows:

$$C_a = F_a A_a \qquad (1.4.2.3\text{-}1)$$

and

$$C_v = F_v A_v \qquad (1.4.2.3\text{-}2)$$

TABLE 1.4.2.3a
Values of F_a as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_a \le 0.1$	$A_a = 0.2$	$A_a = 0.3$	$A_a = 0.4$	$A_a \ge 0.5^a$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*b*
F	*b*	*b*	*b*	*b*	*b*

NOTE: Use straight line interpolation for intermediate values of A_a.

[a] Values for $A_a > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

[b] Site specific geotechnical investigation and dynamic site response analyses shall be performed.

TABLE 1.4.2.3b
Values of F_v as a Function of Site Conditions and Shaking Intensity

Soil Profile Type	Shaking Intensity				
	$A_v \le 0.1$	$A_v = 0.2$	$A_v = 0.3$	$A_v = 0.4$	$A_v \ge 0.50^a$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*b*
F	*b*	*b*	*b*	*b*	*b*

NOTE: Use straight line interpolation for intermediate values of A_v.

[a] Values for $A_v > 0.4$ are applicable to the provisions for seismically isolated structures in Sec. 2.6 and certain other structures (e.g., see Table 2.2.4.3).

[b] Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

1.4.2.4 Seismic Coefficients C_a and C_v: Seismic coefficient C_a based on Soil Profile Type and A_a is determined from Table 1.4.2.4a:

TABLE 1.4.2.4a
Seismic Coefficient C_a

Soil Profile Type	$A_a < 0.05$	$A_a = 0.05$	$A_a = 0.10$	$A_a = 0.20$	$A_a = 0.30$	$A_a = 0.40$
A	A_a	0.04	0.08	0.16	0.24	0.32
B	A_a	0.05	0.10	0.20	0.30	0.40
C	A_a	0.06	0.12	0.24	0.33	0.40
D	A_a	0.08	0.16	0.28	0.36	0.44
E	A_a	0.13	0.25	0.34	0.36	0.36

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_a.

Seismic coefficient C_v based on Soil Profile Type and A_v is determined from Table 1.4.2.4b:

TABLE 1.4.2.4b
Seismic Coefficient C_v

Soil Profile Type	$A_v < 0.05$	$A_v = 0.05$	$A_v = 0.10$	$A_v = 0.20$	$A_v = 0.30$	$A_v = 0.40$
A	A_v	0.04	0.08	0.16	0.24	0.32
B	A_v	0.05	0.10	0.20	0.30	0.40
C	A_v	0.09	0.17	0.32	0.45	0.56
D	A_v	0.12	0.24	0.40	0.54	0.64
E	A_v	0.18	0.35	0.64	0.84	0.96

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determine the value of C_v.

Note that where A_a and A_v are less than 0.05, $C_a = A_a$ and $C_v = A_v$.

1.4.3 SEISMIC HAZARD EXPOSURE GROUPS: All buildings shall be assigned to one of the following Seismic Hazard Exposure Groups:

1.4.3.1 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

1.4.3.2 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

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

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

1.4.3.5 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 ft (3 m) 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.

1.4.3.6 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. 3.3.8 that could result in the interruption of utility services shall be considered when providing the capacity to continue to function.

1.4.4 SEISMIC PERFORMANCE CATEGORY: Buildings shall be assigned a Seismic Performance Category in accordance with Table 1.4.4.

TABLE 1.4.4
Seismic Performance Category

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

1.4.5 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 ALTERNATIVE MATERIALS AND METHODS OF CONSTRUCTION: Alternative materials and methods of construction to those prescribed in these provisions are permitted if approved by 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 seismic-force-resisting and other designated seismic systems. These requirements supplement the testing and inspection requirements contained in the reference standards given in Chapters 3 through 9. As a minimum, the quality assurance provisions apply to the following:

1. The seismic-force-resisting systems in buildings assigned to Seismic Performance Categories C, D, and E.

2. Other designated seismic systems in buildings assigned to Seismic Performance Categories D and E.

1.6.1 QUALITY ASSURANCE PLAN: A quality assurance plan (QAP) 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 seismic-force-resisting or other designated seismic system or component listed in the QAP 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 QAP;

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; and

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

1.6.2 SPECIAL INSPECTION: The owner shall employ a special inspector(s) to observe the construction of the seismic-force-resisting and all other designated seismic systems in accordance with the QAP for the following construction work:

1.6.2.1 Foundations: Continuous special inspection during driving of piles and placement of concrete. Periodic special inspection during construction of drilled piles and piers and caissons and the placement of reinforcing steel.

1.6.2.2 Reinforcing Steel:

1.6.2.2.1: Periodic special inspection during and upon completion of the placement of reinforcing steel in intermediate and special moment frames of concrete and concrete shear walls.

1.6.2.2.2: Continuous special inspection during the welding of reinforcing steel resisting flexural and axial forces in intermediate and special moment frames of concrete, in boundary members of concrete shear walls, and welding of shear reinforcement.

1.6.2.3 Structural Concrete: Periodic special inspection during and on completion of the placement of concrete in intermediate and special moment frames and in boundary members of concrete shear walls.

1.6.2.4 Prestressed Concrete: Periodic special inspection during the placement and after completion of 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:

1.6.2.5.1: Periodic special inspection during the preparation of mortar, the laying of masonry units, and placement of reinforcement and prior to placement of grout

1.6.2.5.2: Continuous special inspection during the welding of reinforcement, grouting, consolidation, and reconsolidation.

1.6.2.6 Structural Steel:

1.6.2.6.1: Continuous special inspection for all structural welding.

> **EXCEPTION:** Periodic special inspection is permitted for single-pass fillet or resistance welds and welds loaded to less than 50 percent of their design strength provided the qualifications of the welder and the welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the approved construction documents at the completion of welding.

1.6.2.6.2: Periodic special inspection in accordance with Ref. 5-1 or 5-2 for installation and tightening of fully tensioned high-strength bolts in slip-critical connections and in connections subject to direct tension. Bolts in connections identified as not being slip-critical or subject to direct tension need not be inspected for bolt tension other than to ensure that the plies of the connected elements have been brought into snug contact.

1.6.2.7 Structural Wood:

1.6.2.7.1: Continuous special inspection during all field gluing operations of elements of the seismic-force-resisting system.

1.6.2.7.2: Periodic special inspection for nailing, bolting, anchoring, and other fastening of all seismic components including drag struts, braces, and hold downs.

1.6.2.8 Architectural Components: Special inspection for architectural components shall be as follows:

1. Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonloadbearing walls, and veneer in Seismic Performance Categories D and E and

   EXCEPTIONS:

   a. Buildings 30 feet (9 m) or less in height and

   b. Cladding and veneer weighing 5 lb/ft^2 (240 kg/m^2) or less.

2. Periodic special inspection during the anchorage of access floors and storage racks 8 feet (2.4 m) or greater in height in Seismic Performance Categories D and E.

1.6.2.9 Mechanical and Electrical Components: Special inspection for mechanical and electrical components shall be as follows:

1. Periodic special inspection during the anchorage of electrical equipment for emergency or standby power systems in Seismic Performance Categories C, D, and E;

2. Periodic special inspection during the installation of anchorage of all other electrical equipment in Seismic Performance Category E;

3. Periodic special inspection during installation for flammable, combustible, or highly toxic piping systems and their associated mechanical units in Seismic Performance Categories C, D, and E; and

4. Periodic special inspection during the installation of HVAC ductwork that will contain hazardous materials in Seismic Performance Categories C, D, and E.

1.6.3 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 the seismic-force-resisting and other 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: Examine certified mill test reports for each shipment of reinforcing steel used to resist flexural and axial forces in reinforced concrete intermediate and special moment frames and boundary members of reinforced concrete or reinforced masonry shear walls and determine conformance with specification requirements.

1.6.3.1.2: Where ASTM A615 reinforcing steel is used to resist earthquake-induced flexural and axial forces in special moment frames and in wall boundary elements of shear walls in buildings of Seismic Performance Category D and E, verify that the requirements of Sec. 21.2.5.1 of Ref. 6-1 have been satisfied.

1.6.3.1.3: Where ASTM A615 reinforcing steel is to be welded, verify that chemical tests have been performed to determine weldability in accordance with Sec. 3.5.2 of Ref. 6-1.

1.6.3.2 Structural Concrete: Samples of structural concrete shall be obtained at the project site and tested in accordance with requirements of Ref. 6-1.

1.6.3.3 Structural Masonry: Quality assurance testing of structural masonry shall be in accordance with the requirements of Ref. 8-1.

1.6.3.4 Structural Steel: Special testing of structural steel shall be as follows:

1.6.3.4.1: Welded connections for moment frames and eccentrically braced frames shall be tested by nondestructive methods conforming to ANSI/AWS D1.1-90, *Structural Welding Code*, Sec. 5 and 6. All complete penetration groove welds contained in joints and splices shall be 100 percent tested either by ultrasonic testing or by other approved methods.

> **EXCEPTION:** The nondestructive testing rate for an individual welder may be reduced to 25 percent with the concurrence of the design professional(s) of record, provided that 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 designed to resist tension resulting from the prescribed seismic design forces shall be tested by ultrasonic testing or other approved methods at a rate established by the design professional(s) of record.

1.6.3.4.3: Base metal thicker than 1.5 in. (38 mm) 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 ASTM A435, *Specification for Straight Beam Ultrasound Examination of Steel Plates*, or ASTM A898, *Specification for Straight Beam Ultrasound Examination for Rolled Steel Shapes*, and criteria as established by the design professional(s) of record and the contract documents.

1.6.3.5 Mechanical and Electrical Equipment: As required to ensure compliance with the seismic design provisions herein, the facility designer shall clearly state the applicable requirements on contract documents. Each manufacturer of these designated components shall test or analyze the component and its mounting system or anchorage as required and 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 shall be by actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance), or by more rigorous analysis providing for equivalent safety. The special inspector shall examine the designated seismic system component and shall determine whether the anchorages and label conform with the certificate of compliance.

1.6.3.6 Seismically Isolated Structures: For required system tests, see Sec. 2.6.9.

1.6.4 REPORTING AND COMPLIANCE PROCEDURES: Each special inspector shall furnish to the regulatory agency, design professional of record, the owner, the persons preparing the QAP, and the contractor copies of regular weekly progress reports of the inspector's 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 construction documents. Work not in compliance shall be described in the final report.

At completion of construction, the building contractor shall submit a final report to the regulatory agency certifying that all construction work incorporated into the seismic-force-resisting and other designated seismic systems was constructed substantially in accordance with the construction documents and applicable workmanship requirements. Work not in compliance shall be described in the final report.

The contractor shall correct all deficiencies as required.

Appendix to Chapter 1

DEVELOPMENT OF DESIGN VALUE MAPS

The BSSC has been working to develop new design values maps to replace the present Maps 1 through 4 (included in the maps packet for the 1994 Edition), which have been in use for nearly 20 years. The BSSC Provisions Update Committee (PUC) has been unable to reach consensus on this issue for this edition of the *NEHRP Recommended Provisions*. This appendix is provided to inform professionals of the work in progress and to solicit comments. As indicated at the conclusion of this appendix, the effort to update these provisions for issuance in 1997 contains a specific task devoted to the development of newer hazard maps and a design value procedure that can achieve consensus approval.

The BSSC has attempted to use as a basis for developing new design values maps the spectral response ordinate acceleration maps developed by the U.S. Geologic Survey (USGS) and included in the *Provisions* as Maps 5 through 8 (included in the maps packet for the 1994 Edition). These are probabilistic-based hazard maps for all areas of the continental United States. The BSSC has rejected their direct use as design values maps since the *Provisions* maps are used to control many items such as triggering design and detailing provisions that have not been scaled to the spectral response ordinate maps. The values of ground motion on Maps 5 through 8 also were believed to be low in some areas of known seismicity and further review has been requested of attenuation functions and of the need for input beyond pure probabilistic hazard as the basis for design values maps.

Most of the controversy regarding Maps 5 through 8 occurred in the western United States where many individual efforts to define seismic hazard are under way. These efforts have led to quite different results in some areas and are illustrated on Maps 13 and 14 (included in the maps packet for the 1994 Edition), which were developed within the PUC's Design Values Panel during the effort to generate this edition of the *Provisions*. These maps are based on seismic hazard data developed by local and regional professionals for their respective areas and may not represent uniform seismic risk. The sources included the structural engineers associations in California and Washington, the geologic surveys of Utah and Idaho, and sources in the other western states.

Map 15 (included in the maps packet for the 1994 Edition), a seismic zoning map for the state of New York, also is included to illustrate current independent hazard mapping. This map was completed in 1993 as part of the technically approved but not yet legally adopted draft for the seismic provisions of the New York State Building Code (NYSBC). It is stepped along county lines to facilitate code administration. It displays four seismic zones (A, B, C, and D) with associated seismic zone factors Z = 0.09, 0.12, 0.15, and 0.18 respectively. The Z numerically corresponds to effective peak acceleration in g on rock/stiff soil (S_1) conditions (shear wave velocities of about 2,500 ft/sec). The NYSBC map is not a probabilistic map but rather represents a consensus of the group responsible for its development. Comparison of this map with recently computed probabilistic ground motion maps indicates that in zones of lowest seismic hazard, it corresponds to 10 percent exceedance probability in 100 years exposure time; in regions of intermediate hazard (Zone C, Z = 0.15), it coincides closely with the 10 percent in 50-year maps; and in Zone D (Z = 0.18), it falls below what is expected from the 50-year maps.

This indicates that, in general, the NYSBC map smoothes out the extreme highs and lows that exist in computed ground motion maps. In short, the NYSBC map does not represent a single uniform hazard level.

Also included for information only are Maps 9 through 12 (included in the maps packet for the 1994 Edition) which are spectral response ordinate acceleration maps for a 90 percent probability of nonexceedance in 250 years. These maps are recommended for any design or basis of seismic criteria without thorough independent study and verification. For example, for long exposure times such as 250 years, the mapped ground motion values depend very heavily on the uncertainty of the inputs to the probabilistic models. Note that both the 50-year maps (Maps 5 through 8) and the 250-year maps (Maps 9 through 12) included with this 1994 Edition of the *Provisions* have been updated by the USGS from the 1991 Edition maps to reflect additional data.

Many requirements in the *NEHRP Recommended Provisions* will have to be revised if spectral ordinate design maps eventually are used. Provisions that would have to change with spectral ordinate design maps were outlined in detail in the "Appendix to Chapter 1" of the 1991 *Provisions* and are not repeated here. For rough scaling, the spectral response values on Maps 5 and 6 are approximately 2.5 times the A_a and A_v values on the present design maps, Maps 1 through 4.

Maps 5 through 12 are similar to the maps in the "Appendix to Chapter 1" of the 1991 *Provisions* with changes limited to northern California and the Pacific Northwest.

A key element in the preparation of seismic hazard maps is the method utilized in attenuating ground motion from a potential seismic source to any particular location. The attenuation relationship utilized in preparing Maps 1 through 4 is based on site conditions characterized as rock and the current site coefficients are anchored to rock to conform to the site conditions for Maps 1 through 4.

The newer spectral ordinate hazard maps (Maps 5 through 12) were developed using an attenuation relationship based on soil rather than rock conditions to permit direct use of a larger database of strong motion instrument readings. Use of these maps or design values maps based on the spectral ordinate hazard maps will require adjustment of the site coefficients to the soil anchor conditions conforming to these maps. The resulting site coefficients would be considerably lower than the current ones. Rock and hard rock sites would have site coefficients of less than one, thus lowering the equivalent lateral force at those sites by up to 60 percent. Such reductions may affect seismic performance categories as presently defined. The soils issue is further complicated because Maps 5 through 12 have been calibrated to the former S_2 soil type whereas the new seismic coefficients adopted for the 1994 *Provisions* do not have a direct translation to the old S_2 soil and therefore modification of the soil factors is required for use of the spectral maps.

Another issue on which consensus was not achieved is that of the near-field condition along major faults. It generally was agreed that a ceiling of $S_{A(0.3)} = 100$ and $S_{A(1.0)} = 60$ was appropriate to reflect the current ceiling of A_a and $A_v = 0.40$. Along active faults capable of generating earthquakes of magnitude 7.5 or greater, special design limitations were proposed. These design limitations included a maximum R value for permitted construction, a prohibition of offsets in plan location of seismic bracing elements, and possible special provisions for essential facilities. As noted, however, consensus was not reached on the provisions or on the applicable faults. The BSSC solicits all opinions and suggestions on this issue.

Further, during the development of the 1997 Edition of the *NEHRP Recommended Provisions*, the BSSC will develop a new, more rational seismic design procedure for use by engineers and architects. Unlike the current design procedure, which uses the relatively dated USGS peak velocity and peak acceleration ground motion maps, the revised procedure will be based on the USGS spectral response maps, which are being revised. It also will take into account the design concerns that were raised by the BSSC Design Values Panel during the 1994 *Provisions* update effort. The new design procedure may take the form of a separate design map based on the updated USGS hazard maps or may involve a process specified within the body of the *Provisions*. In developing the procedure, the BSSC plans to utilize a process that includes a mechanism to allow for public input. As part of this task, the BSSC has developed and signed a Memorandum of Understanding with the USGS that specifies that the USGS will hold a series of regional workshops on the current seismic hazard maps and will document the results of those workshops, including comments received and how they were addressed. Based on the results of those workshops, the USGS will revise its seismic hazard maps to reflect the state of the art. The BSSC then will mount the effort to develop the needed design procedure using the new hazard maps as a starting point. Conduct of this task will involve a five-member Management Committee to provide overall guidance and a Resource Group consisting of representatives of interested organizations from the design, construction, and earth sciences communities. A 12-member Seismic Design Procedure Group will be responsible for development of the actual procedure. As part of the process of developing the procedure, the BSSC plans to conduct five regional workshops to solicit, examine, and resolve regional issues related to the development of the design procedure and to begin to obtain consensus on the framework of the design procedure. Upon completion of the draft design procedure, the group will submit it to the BSSC 1997 Provisions Update Committee for inclusion in the ballot package of the 1997 Edition of the *Provisions*. Any readers interested in this development effort are urged to contact the BSSC.

Chapter 2

STRUCTURAL DESIGN CRITERIA, ANALYSIS, AND PROCEDURES

2.1 REFERENCE DOCUMENT

The following reference document shall be used for loads other than earthquakes and for combinations of loads as indicated in this chapter:

Ref. 2-1 *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-93

2.2 STRUCTURAL DESIGN REQUIREMENTS:

2.2.1 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 Sec. 2.3 or Sec. 2.4, 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 energy dissipation capacity of the structure.

2.2.2 STRUCTURAL FRAMING SYSTEMS: The basic structural framing systems to be used are indicated in Table 2.2.2. 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 2.2.2. The appropriate response modification coefficient, R, and the deflection amplification factor, C_d, indicated in Table 2.2.2 shall be used in determining the base shear and design story drift.

TABLE 2.2.2
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 (ft) 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
Plain (unreinforced) masonry shear walls	1¼	1¼	NL	*f*	NP	NP
Plain concrete shear walls	1½	1½	NL	*g*	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
Composite eccentrically braced frames (C-EBF)	8	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	100
Composite concentrically braced frames (C-CBF)	5	4½	NL	NL	160	100
Special concentrically braced frame of steel	6	5	NL	NL	160	100
Reinforced concrete shear walls	5½	5	NL	NL	160	100
RC shear walls composite with steel elements	5½	5	NL	NL	160	100
Steel plate reinforced composite shear walls	6½	5½	NL	NL	160	100
Reinforced masonry shear walls	4½	4	NL	NL	160	100
Plain (unreinforced) masonry shear walls	1½	1½	NL	*f*	NP	NP
Plain concrete shear walls	2	2	NL	*g*	NP	NP

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 (ft) Limitations[c] — Seismic Performance Category: A & B	C	D[d]	E[e]
Moment Resisting Frame System						
Special moment frames of steel	8	5½	NL	NL	NL	NL
Special moment frames of reinforced concrete	8	5½	NL	NL	NL	NL
Special moment frame of masonry	5½	5	NL	NL	160	160
Composite special moment frame (C-SMF)	8	5½	NL	NL	NL	NL
Intermediate moment frames of reinforced concrete	5	4½	NL	NL	NP	NP
Ordinary moment frames of steel	4½	4	NL	NL	160	100
Composite ordinary moment frame (C-OMF)	4½	4	NL	NL	160	100
Composite partially restrained frames (C-PRF)	6	5½	160	160	100	NP
Ordinary moment frames of reinforced concrete	3	2½	NL[h]	NP	NP	NP
Dual System with a Special Moment Frame Capable of Resisting at Least 25% of Prescribed Seismic Forces						
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
Composite eccentrically braced frames (C-EBF)	8	4	NL	NL	NL	NL
Concentrically braced frames	6	5	NL	NL	NL	NL
Special concentrically braced frames of steel	8	6½	NL	NL	NL	NL
Composite concentrically braced frames (C-CBF)	6	5	NL	NL	NL	NL
Reinforced concrete shear walls	8	6½	NL	NL	NL	NL
RC shear walls composite with steel elements	8	6½	NL	NL	NL	NL
Steel plate reinforced composite 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

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 (ft) Limitations[c] — Seismic Performance Category			
			A & B	C	D[d]	E[e]
<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
Composite concentrically braced frames (C-CBF)	5	4½	NL	NL	160	100
Special concentrically braced frame of steel	6	5	NL	NL	160	100
Reinforced concrete shear walls	6	5	NL	NL	160	100
RC shear walls composite with steel elements	6	5	NL	NL	160	100
Steel plate reinforced composite shear walls	7	5½	NL	NL	160	100
Reinforced masonry shear walls	5	4½	NL	NL	160	100
Wood sheathed shear panels	7	4½	NL	NL	160	100
<u>Inverted Pendulum Structures--Seismic Force Resisting System</u>						
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 throughout the *Provisions.*
[b] Deflection amplification factor, C_d, for use in Sec. 2.3.7.1 and 2.3.7.2.
[c] NL = Not Limited and NP = Not Permitted. If using metric units, 100 ft approximately equals 30 m and 160 ft approximately equals 50 m.
[d] See Sec. 2.2.2.4.1 for a descriptions of building systems limited to buildings with a height of 240 ft (70 m) or less.
[e] See Sec. 2.2.2.5 for building systems limited to buildings with a height of 160 ft (50 m) or less.
[f] The masonry shear walls shall have nominal reinforcement as required by Sec. 8.3.7.2.
[g] Plain concrete shear walls shall have nominal reinforcement in accordance with Sec. 8.3.7.2.
[h] Ordinary moment frames of reinforced concrete are not permitted as a part of the seismic force resisting system in Seismic Performance Category B buildings founded on Soil Profile Types E or F (see Sec. 6.5.2).

Structural framing and resisting systems that are not contained in Table 2.2.2 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 2.2.2 for equivalent response modification coefficient, R, values.

Special framing requirements are indicated in Sec. 2.2.5 and in Chapters 5, 6, 7, 8, and 9 for buildings assigned to the various seismic performance categories.

2.2.2.1 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.

2.2.2.2 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.

2.2.2.2.1 Combination Framing Factor: The response modification coefficient, 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.

2.2.2.2.2 Combination Framing Detailing Requirements: The detailing requirements of Sec. 2.2.5 required by the higher response modification coefficient, R, shall be used for structural components common to systems having different response modification coefficients.

2.2.2.3 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 2.2.2.

2.2.2.4 Seismic Performance Category D: The structural framing system for a building assigned to Seismic Performance Category D shall comply with Sec. 2.2.2.3 and the additional provisions of this section.

2.2.2.4.1 Limited Building Height: The height limits in Table 2.2.2 may be increased to 240 ft (70 m) 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.

2.2.2.4.2 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. 2.3.7.

2.2.2.4.3 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. 2.3.7 (also see Sec. 2.2.7).

2.2.2.4.4 Special Moment Frames: A special moment frame that is used but not required by Table 2.2.2 is permitted to be discontinued and supported by a more rigid system with a lower response modification coefficient, R, provided the requirements of Sec. 2.2.5.2.4 and 2.2.5.4.2 are met. Where a special moment frame is required by Table 2.2.2, the frame shall be continuous to the foundation.

2.2.2.5 Seismic Performance Category E: The framing systems of buildings assigned to Category E shall conform to the requirements of Sec. 2.2.2.4 for Category D and to the additional requirements and limitations of this section. The height limitation of Sec. 2.2.2.4.1 shall be reduced from 240 ft to 160 ft (70 to 50 m).

2.2.3 BUILDING CONFIGURATION: Buildings shall be classified as regular or irregular based upon the criteria in this section. Such classification shall be based on the plan and vertical configuration.

2.2.3.1 Plan Irregularity: Buildings having one or more of the features listed in Table 2.2.3.1 shall be designated as having plan structural irregularity and shall comply with the requirements in the sections referenced in Table 2.2.3.1.

2.2.3.2 Vertical Irregularity: Buildings having one or more of the features listed in Table 2.2.3.2 shall be designated as having vertical irregularity and shall comply with the requirements in the sections referenced in Table 2.2.3.2.

EXCEPTIONS:

1. Structural irregularities of Types 1 or 2 in Table 2.2.3.2 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 2.2.3.2 are not required to be considered for 1- and 2-story buildings.

TABLE 2.2.3.1
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.	2.2.5.4.2 2.3.5.1	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.	2.2.5.4.2	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.	2.2.5.4.2	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.	2.2.5.4.2	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.	2.2.5.3.1	C, D, and E

TABLE 2.2.3.2
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.	2.2.4.3	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.	2.2.4.3	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.	2.2.4.3	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.	2.2.5.4.2	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.	2.2.5.2.4 and 2.2.5.4.2	B, C, D, and E

2.2.4 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 Sec. 2.4 or, with the approval of the regulatory agency, an alternate generally accepted procedure, including the use of an approved site-specific spectrum, is permitted for any building. The limitations on the base shear stated in Sec. 2.4 apply to dynamic modal analysis.

2.2.4.1 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. 2.2.5.1 apply.

2.2.4.2 Seismic Performance Categories B and C: The analysis procedures in Sec. 2.3 shall be used for regular or irregular buildings assigned to Category B or C or a more rigorous analysis may be made.

2.2.4.3 Seismic Performance Categories D and E: The analysis procedures identified in Table 2.2.4.3 shall be used for buildings assigned to Categories D and E or a more rigorous analysis may be made.

2.2.5 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 4. The materials and the systems composed of those materials shall conform to the requirements and limitations of Chapters 5 through 9 for the applicable category.

2.2.5.1 Seismic Performance Category A: The design and detailing of buildings assigned to Category A shall comply with the requirements of this section.

2.2.5.1.1 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 seismic coefficient, C_a, 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.

2.2.5.1.2 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 seismic coefficient C_a in pounds per lineal foot (14,600 times C_a in N/m) of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1.2 m).

TABLE 2.2.4.3
Analysis Procedures for Seismic Performance Categories D and E

Building Description	Reference and Procedures
1. Buildings designated as regular up to 240 ft (70 m)	Sec. 2.3
2. Buildings that have only vertical irregularities of Type 1, 2, or 3 in Table 2.2.3.2 and have a height exceeding 5 stories or 65 ft (20 m)and all buildings exceeding 240 ft (70 m) in height	Sec. 2.4
3. All other buildings designated as having plan or vertical irregularities	Sec. 2.3 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 km) 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. 2.3.2
5. Buildings in areas with A_v of 0.2 and greater with a period of 0.7 seconds or greater located on Soil Profile Type E sites	A site-specific response spectrum shall be used but the design base shear shall not be less than that determined from Sec. 2.3.2; also, the modal seismic design coefficient, C_{sm}, shall not be limited per Sec. 2.4.5

2.2.5.1.3 Anchorage of Nonstructural Systems: When required by Chapter 3, all portions or components of the building shall be anchored for the seismic force, F_p, prescribed therein.

2.2.5.2 Seismic Performance Category B: Buildings assigned to Category B shall conform to the requirements of Sec. 2.2.5.1 for Category A and the requirements of this section.

2.2.5.2.1 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. 2.2.6. 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.

2.2.5.2.2 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.

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

2.2.5.2.4 Discontinuities in Vertical System: Buildings with a discontinuity in lateral capacity, vertical irregularity Type 5 as defined in Table 2.2.3.2, shall not be over 2 stories or 30 ft (9 m) 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 Sec. 2.3.

2.2.5.2.5 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.

2.2.5.2.6 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.

2.2.5.2.7 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 of the seismic coefficient C_a 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 or welded type connections.

2.2.5.2.8 Bearing Walls: Exterior and interior bearing walls and their anchorage shall be designed for a force equal to the seismic coefficient C_a 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.

2.2.5.2.9 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 Sec. 2.3 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

2.2.5.3 Seismic Performance Category C: Buildings assigned to Category C shall conform to the requirements of Sec. 2.2.5.2 for Category B and to the requirements of this section.

2.2.5.3.1 Plan Irregularity: Buildings that have plan structural irregularity Type 5 in Table 2.2.3.1 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.

2.2.5.4 Seismic Performance Categories D and E: Buildings assigned to Category D or E shall conform to the requirements of Sec. 2.2.5.3 for Category C and to the requirements of this section.

2.2.5.4.1 Orthogonal Load Effects: Buildings shall be designed for the critical load effect due to application of seismic forces. The alternative procedure in Sec. 2.2.5.3.1 may be used.

2.2.5.4.2 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 2.2.3.1 or a vertical structural irregularity of Type 4 in Table 2.2.3.2, the design forces determined from Sec. 2.3.2 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements.

2.2.5.4.3 Vertical Seismic Forces: The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. The load combinations used in evaluating such components shall include E as defined by Eq. 2.2.6-4. 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. 2.2.6.

2.2.6 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 factored load combinations as presented in ANSI/ASCE 7 (Ref. 2-1) except that the effect of seismic loads, E, shall be as defined herein.

The effect of seismic load E shall be defined by Eq. 2.2.6-1 as follows for load combinations in which the effects of gravity and seismic loads are additive:

$$E = Q_E + 0.5C_aD \qquad (2.2.6\text{-}1)$$

where:

E = the effect of horizontal and vertical earthquake-induced forces,

C_a = the seismic coefficient based upon the Soil Profile Type and the value of A_a as determined from Sec. 1.4.2.3 or Table 1.4.2.4a.

D = the effect of dead load, and

Q_E = the effect of horizontal seismic forces.

The effect of seismic load E shall be defined by Eq. 2.2.6-2 as follows for load combinations in which the effects of gravity counteract seismic load:

$$E = Q_E - 0.5C_aD \qquad (2.2.6\text{-}2)$$

where E, Q_E, C_a, and D are as defined above.

The load factor on E need not be taken as greater than 1.0 in factored load combinations when using the seismic loads as defined in these provisions.

For columns supporting discontinuous lateral-force-resisting elements, the axial force in the columns shall be computed using the most critical load combinations. Load combinations including seismic loads also shall be investigated except the effect of seismic load E shall be defined by Eq. 2.2.6-3 as follows:

$$E = \left(\frac{2R}{5}\right)Q_E + 0.5C_aD \qquad (2.2.6\text{-}3)$$

where E, Q_E, C_a, and D are as defined above and R is the response modification coefficient as given in Table 2.2.2. The axial forces in such columns need not exceed the capacity of other elements of the structure to transfer such loads to the column.

Brittle materials, systems, and connections shall be designed using the most critical load combinations. Load combinations including seismic loads also shall be investigated except the effect of seismic load E shall be defined by Eq. 2.2.6-4 as follows:

$$E = \left(\frac{2R}{5}\right)Q_E - 0.5C_aD \qquad (2.2.6\text{-}4)$$

where E, R, Q_E, C_a, and D are as defined above.

The factor $(2R/5)$ shall be equal to or greater than 1.0. The term $0.5C_a$ is permitted to be neglected where A_v is equal to or less than 0.05.

2.2.7 DEFLECTION AND DRIFT LIMITS: The design story drift, Δ, as determined in Sec. 2.3.7 or 2.4.6, shall not exceed the allowable story drift, Δ_a, as obtained from Table 2.2.7 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. 2.3.7.1.

TABLE 2.2.7
Allowable Story Drift, Δ_a [a] (in. or mm)

Building	Seismic Hazard Exposure Group		
	I	II	III
Buildings, other than masonry shear wall or masonry wall frame buildings, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025\ h_{sx}$ [b]	$0.020\ h_{sx}$	$0.015\ h_{sx}$
Masonry cantilever shear wall buildings [c]	$0.010\ h_{sx}$	$0.010\ h_{sx}$	$0.010\ h_{sx}$
Other masonry shear wall buildings	$0.007\ h_{sx}$	$0.007\ h_{sx}$	$0.007\ h_{sx}$
Masonry wall frame buildings	$0.013\ h_{sx}$	$0.013\ h_{sx}$	$0.010\ h_{sx}$
All other buildings	$0.020\ h_{sx}$	$0.015\ h_{sx}$	$0.010\ h_{sx}$

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

[b] There shall be no drift limit for single-story buildings with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.

[c] Buildings in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

2.3 EQUIVALENT LATERAL FORCE PROCEDURE:

2.3.1 GENERAL: This section 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. 2.2.4 for limitations on the use of this procedure.

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

$$V = C_s W \tag{2.3.2}$$

where:

C_s = the seismic response coefficient determined in accordance with Sec. 2.3.2.1 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 psf (500 Pa/m^2) of floor area, whichever is greater, shall be applicable.

3. Total operating weight of permanent equipment.

2.3.2.1 Calculation of Seismic Response Coefficient: When the fundamental period of the building is computed, the seismic response coefficient, C_s, shall be determined in accordance with the following equation:

$$C_s = \frac{1.2C_v}{RT^{2/3}} \tag{2.3.2.1-1}$$

where:

C_v = the seismic coefficient based upon the Soil Profile Type and the value of A_v as determined from Sec. 1.4.2.3 or Table 1.4.2.4b,

R = the response modification factor from Table 2.2.2, and

T = the fundamental period of the building (sec) determined in Sec. 2.3.3.

A soil-structure interaction reduction is permitted when determined using Sec. 2.5 or other generally accepted procedures approved by the regulatory agency.

Alternatively, the seismic response coefficient need not be greater than the following:

$$C_s = \frac{2.5C_a}{R} \tag{2.3.2.1-2}$$

where R is as above and C_a = the seismic coefficient based upon the Soil Profile Type and the value of A_a as determined from Sec. 1.4.2.3 or Table 1.4.2.4a.

2.3.3 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

*The live load may be reduced for tributary area as permitted by the building code administered by the regulatory agency.

period, T, shall not exceed the product of the coefficient for upper limit on calculated period, C_u, from Table 2.3.3 and the approximate fundamental period, T_a, determined from the appropriate requirements of Sec. 2.3.3.1.

TABLE 2.3.3
Coefficient for Upper Limit on Calculated Period

Seismic Coefficient C_v	Coefficient C_u
≥ 0.4	1.2
0.3	1.3
0.2	1.4
0.15	1.5
0.1	1.7
0.05	1.7

2.3.3.1 Approximate Fundamental Period: The approximate fundamental period, T_a, in seconds, shall be determined from the following equation:

$$T_a = C_T h_n^{3/4}$$

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 (the metric coefficient is 0.0853),

$C_T =$ 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 (the metric coefficient is 0.0731),

$C_T =$ 0.030 for eccentrically braced steel frames (the metric coefficient is 0.0731),

$C_T =$ 0.020 for all other building systems (the metric coefficient is 0.0488), and

$h_n =$ the height (ft or m) 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 ft (3 m):

$$T_a = 0.1N \qquad (2.3.3.1\text{-}2)$$

where N = number of stories.

2.3.4 VERTICAL DISTRIBUTION OF SEISMIC FORCES: The lateral force, F_x (kip or kN), induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \qquad (2.3.4\text{-}1)$$

and

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n}} w_i h_i^k \qquad (2.3.4\text{-}2)$$

where:

C_{vx} = vertical distribution factor,

V = total design lateral force or shear at the base of the building (kip or kN),

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 (ft or m) 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

2.3.5 HORIZONTAL SHEAR DISTRIBUTION: The seismic design story shear in any story, V_x (kip or kN), shall be determined from the following equation:

$$V_x = \sum_{i=1}^{x} F_i \tag{2.3.5}$$

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

The seismic design story shear, V_x (kip or kN), 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.

2.3.5.1 Torsion: The design shall include the torsional moment, M_t (kip·ft or kN·m), resulting from the location of the building masses plus the accidental torsional moments, M_{ta} (kip·ft or kN·m), 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 2.2.3.1 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:

$$A_x = \left(\frac{\delta_{max}}{1.2\,\delta_{avg}} \right)^2 \tag{2.3.5.1}$$

where:

δ_{max} = the maximum displacement at Level x (in. or mm) and

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x (in. or mm).

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.

2.3.6 OVERTURNING: The building shall be designed to resist overturning effects caused by the seismic forces determined in Sec. 2.3.4. 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 (kip·ft or kN·m), shall be determined from the following equation:

$$M_x = \tau \sum_{i=x}^{n} F_i (h_i = h_x) \tag{2.3.6}$$

where:

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

h_i and h_x = the height (ft or m) 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 (kip·ft or kN·m), at the foundation-soil interface determined using the equation for the overturning moment at Level x, M_x (kip·ft or kN·m), with an overturning moment reduction factor, τ, of 0.75 for all building heights.

2.3.7 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.

2.3.7.1 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 (in. or mm), shall be determined in accordance with following equation:

$$\delta_x = C_d \delta_{xe} \tag{2.3.7.1}$$

where:

C_d = the deflection amplification factor in Table 2.2.2 and

δ_{xe} = the deflections determined by an elastic analysis (in. or mm).

The elastic analysis of the seismic force resisting system shall be made using the prescribed seismic design forces of Sec. 2.3.4.

For determining compliance with the story drift limitation of Sec. 2.2.7, the deflections of Level x at the center of mass, δ_x (in. or mm), 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, in seconds, of the building without the upper bound limitation specified in Sec. 2.3.3 when determining drift level seismic design forces.

Where applicable, the design story drift, Δ (in. or mm), shall be increased by the incremental factor relating to the P-delta effects as determined in Sec. 2.3.7.2.

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{2.3.7.2-1}$$

where:

P_x = the total vertical design load at and above Level x (kip or kN); 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 occurring simultaneously with V_x (in. or mm);

V_x = the seismic shear force acting between Level x and x - 1 (kip or kN);

h_{sx} = the story height below Level x (in. or mm); and

C_d = the deflection amplification factor in Table 2.2.2.

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

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{2.3.7.2-2}$$

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. 2.3.7.1 shall be multiplied by 1.0/(1 - θ).

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

2.4 MODAL ANALYSIS PROCEDURE:

2.4.1 GENERAL: This chapter provides required standards for the modal analysis procedure of seismic analysis of buildings. See Sec. 2.2.4 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 Sec. 2.3, with the subscript m denoting quantities in the m^{th} mode.

2.4.2 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.

2.4.3 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.

2.4.4 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.

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

$$V_m = C_{sm} \overline{W}_m \tag{2.4.5-1}$$

$$\overline{W}_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im} \right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \tag{2.4.5-2}$$

where:

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

$\overline{W}_m$ = 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 response coefficient, C_{sm}, shall be determined in accordance with the following equation:

$$C_{sm} = \frac{1.2\, C_v}{R T_m^{2/3}} \tag{2.4.5-3}$$

where:

C_v = The seismic coefficient based upon the Soil Profile Type and the value A as determined from Sec. 1.4.2.3 or Table 1.4.2.4b.

R = the response modification factor determined from Table 2.2.2, 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 C_a divided by the response modification factor, R.

EXCEPTIONS:

1. The limiting value is not applicable to Seismic Performance Category D and E buildings with a period of 0.7 seconds or greater located on Soil Profile Type E or F sites.

2. For buildings on sites with Soil Profile Type D, E, or F, 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:

$$C_{sm} = \frac{C_a}{R}(1.0 + 5.0T_m) \tag{2.4.5-4}$$

   where R and T_m are as defined above and C_a is the seismic coefficient based upon the Soil Profile type and the value of A_a as determined from Sec. 1.4.2.3 or Table 1.4.2.4a.

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:

$$C_{sm} = \frac{3C_v}{RT_m^{4/3}} \tag{2.4.5-5}$$

   where R and T_m are as defined above and C_v is the seismic coefficient based upon the Soil Profile Type and the value of A_v as determined from Sec. 1.4.2.3 or Table 1.4.2.4b.

The reduction due to soil-structure interaction as determined in Sec. 2.5.3 may be used.

2.4.6 MODAL FORCES, DEFLECTIONS, AND DRIFTS: The modal force, F_{xm}, at each level shall be determined by the following equations:

$$F_{xm} = C_{vxm}V_m \tag{2.4.6-1}$$

and

$$C_{vxm} = \frac{w_x\phi_{xm}}{\sum_{i=1}^{n} w_i\phi_{im}} \tag{2.4.6-2}$$

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, 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:

$$\delta_{xm} = C_d \delta_{xem} \tag{2.4.6-3}$$

and

$$\delta_{xem} = \left(\frac{g}{4\pi^2}\right)\left(\frac{T_m^2 F_{xm}}{w_x}\right) \tag{2.4.6-4}$$

where:

C_d = the deflection amplification factor determined from Table 2.2.2,

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 (ft/s^2 or m/s^2),

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.

2.4.7 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. 2.4.6 shall be computed for each mode by linear static methods.

2.4.8 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. 2.4.6 and 2.4.7. 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 Sec. 2.3 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_u, 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:

$$\frac{V}{V_t} \tag{2.4.8}$$

where:

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Sec. 2.3 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 Sec. 2.3.

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 Soil Profile Type E or F site, the design base shear shall not be less than that determined using the equivalent lateral force procedure in Sec. 2.3 (see Sec. 2.2.4.3).

2.4.9 HORIZONTAL SHEAR DISTRIBUTION: The distribution of horizontal shear shall be in accordance with the requirements of Sec. 2.3.5.

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

2.4.11 P-DELTA EFFECTS: The P-delta effects shall be determined in accordance with Sec. 2.3.7.2. The story drifts and story shears shall be determined in accordance with Sec. 2.3.7.1.

2.5 SOIL-STRUCTURE INTERACTION EFFECTS:

2.5.1 GENERAL: The provisions set forth in this section 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. 2.5.2 and those for use with the modal analysis procedure are given in Sec. 2.5.3.

2.5.2 EQUIVALENT LATERAL FORCE PROCEDURE: The following provisions are supplementary to those presented in Sec. 2.3.

2.5.2.1 Base Shear: To account for the effects of soil-structure interaction, the base shear, V, determined from Eq. 2.3.2-1 may be reduced to:

$$\tilde{V} = V - \Delta V \tag{2.5.2.1-1}$$

The reduction, ΔV, shall be computed as follows:

$$\Delta V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\beta} \right)^{0.4} \right] \overline{W} \tag{2.5.2.1-2}$$

where:

C_s = the seismic response coefficient computed from Eq. 2.3.2.1-1 using the fundamental natural period of the fixed base structure (T or T_a) as specified in Sec. 2.3.3,

$\tilde{C}_s$ = the seismic response coefficient computed from Eq. 2.3.2.1-1 using the fundamental natural period of the flexibly supported structure ($\tilde{T}$) defined in Sec. 2.5.2.1.1,

$\tilde{\beta}$ = the fraction of critical damping for the structure-foundation system determined in Sec. 2.5.2.1.2, 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 $0.7V$.

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

$$\tilde{T} = T \sqrt{1 + \frac{\bar{k}}{K_y} \left(1 + \frac{K_y \bar{h}^2}{K_\theta} \right)} \tag{2.5.2.1.1-1}$$

where:

T = the fundamental period of the building as determined in Sec. 2.3.3;

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

$$\bar{k} = 4\pi^2\left(\frac{\bar{W}}{gT^2}\right) \qquad (2.5.2.1.1\text{-}2)$$

$\bar{h}$ = 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 2.5.2.1.1 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 2.5.2.1.1
Values of G/G_o and v_s/v_{so}

	Ground Acceleration Coefficient, A_v			
	≤ 0.10	≤ 0.15	≤ 0.20	≥ 0.30
Value of G/G_o	0.81	0.64	0.49	0.42
Value of V_s/v_{so}	0.90	0.80	0.70	0.65

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

$$\tilde{T} = T\sqrt{1 + \frac{25\alpha r_a \bar{h}}{v_s^2 T^2}\left(1 + \frac{1.12 r_a \bar{h}^2}{r_m^3}\right)} \qquad (2.5.2.1.1\text{-}3)$$

where:

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

$$\alpha = \frac{\overline{W}}{\gamma A_o \bar{h}} \qquad (2.5.2.1.1\text{-}4)$$

r_a and r_m = characteristic foundation lengths defined by:

$$R_a = \sqrt{\frac{A_o}{\pi}} \qquad (2.5.2.1.1\text{-}5)$$

and

$$r_m = \sqrt[4]{\frac{4I_o}{\pi}} \qquad (2.5.2.1.1\text{-}6)$$

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.

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

$$\tilde{\beta} = \beta_o + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \qquad \{2.5.2.1.2\text{-}1)$$

where:

β_o = the foundation damping factor as specified in Figure 2.5.2.1.2.

The values of β_o corresponding to A_v = 0.15 in Figure 2.5.2.1.2 shall be determined by averaging the results obtained from the solid lines and the dashed lines.

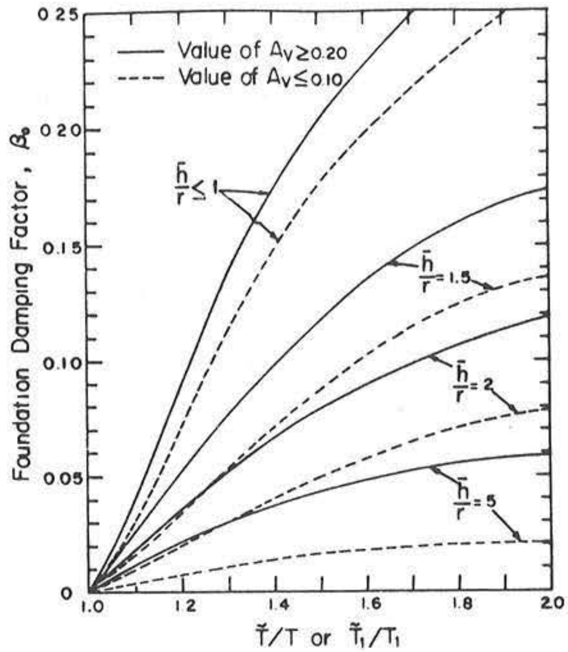

FIGURE 2.5.2.1.2 Foundation damping factor.

The quantity r in Figure 2.5.2.1.2 is a characteristic foundation length that shall be determined as follows:

For $h/L_o \leq 0.5$,

$$r = r_a = \sqrt{\frac{A_o}{\pi}} \tag{2.5.2.1.2-2}$$

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

$$r = r_m = \sqrt[4]{\frac{4I_o}{\pi}} \tag{2.5.2.1.2-3}$$

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. 2.5.2.1.2-1 shall be replaced by:

$$\beta_o' = \left(\frac{4D_s}{V_s\tilde{T}}\right)^2 \beta_o \qquad (2.5.2.1.2\text{-}4)$$

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. 2.5.2.1.2-1, both with or without the adjustment represented by Eq. 2.5.2.1.2-4, shall in no case be taken as less than $\tilde{\beta} = 0.05$.

2.5.2.2 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.

2.5.2.3 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:

$$\tilde{\delta}_x = \frac{\tilde{V}}{V}\left(\frac{M_o h_x}{K_\theta} + \delta_x\right) \qquad (2.5.2.3)$$

where:

M_o = the overturning moment at the base determined in accordance with Sec. 2.3.6 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. 2.3.7.1 using the unmodified seismic forces.

The modified story drifts and P-delta effects shall be evaluated in accordance with the provisions of Sec. 2.3.7 using the modified story shears and deflections determined in this section.

2.5.3 MODAL ANALYSIS PROCEDURE: The following provisions are supplementary to those presented in Sec. 2.4.

2.5.3.1 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:

$$\tilde{V}_1 = V_1 - \Delta V_1 \qquad (2.5.3.1\text{-}1)$$

The reduction, ΔV_1, shall be computed in accordance with Eq. 2.5.2.1-2 with $\overline{W}$ taken as equal to the gravity load $\overline{W}_1$ defined by Eq. 2.4.5-2, C_s computed from Eq. 2.4.5-3 using the fundamental period of the fixed base building, T_1, and $\tilde{C}_s$ computed from Eq. 2.4.5-3 using the fundamental period of the elastically supported building, $\tilde{T}_1$.

The period $\tilde{T}_1$ shall be determined from Eq. 2.5.2.1.1-1, or from Eq. 2.5.2.1.1-3 when applicable, taking $T = T_1$, evaluating $\overline{k}$ from Eq. 2.5.2.1.1-2 with $\overline{W} = \overline{W}_1$, and computing $\overline{h}$ as follows:

$$\overline{h} = \frac{\sum_{i=1}^{n} w_i \phi_{i1} h_i}{\sum_{i=1}^{n} w_i \phi_{i1}} \qquad (2.5.3.1\text{-}2)$$

The above designated values of $\overline{W}$, $\overline{h}$, T, and $\tilde{T}$ also shall be used to evaluate the factor α from Eq. 2.5.2.1.1-4 and the factor β_o from Figure 2.5.2.1.2. No reduction shall be made in the shear components contributed by the higher modes of vibration. The reduced base shear, $\tilde{V}_1$, shall in no case be taken less than $0.7V_1$.

2.5.3.2 Other Modal Effects: The modified modal seismic forces, story shears, and overturning moments shall be determined as for buildings without interaction using the modified base shear, $\tilde{V}_1$, instead of V_1. The modified modal deflections, $\tilde{\delta}_{xm}$, shall be determined as follows:

$$\tilde{\delta}_{x1} = \frac{\tilde{V}_1}{V_1}\left[\frac{M_{o1} h_x}{K_\theta} + \delta_{x1}\right] \qquad (2.5.3.2\text{-}1)$$

and

$$\tilde{\delta}_{xm} = \delta_{xm} \text{ for } m = 2, 3, ... \qquad 2.5.3.2\text{-}2)$$

where:

M_{o1} = the overturning base moment for the fundamental mode of the fixed-base building, as determined in Sec. 2.4.7 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. 2.4.6 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.

2.5.3.3 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. 2.3.5 and the *P*-delta effects shall be evaluated in accordance with the provisions of Sec. 2.3.7.2, using the story shears and drifts determined in Sec. 2.5.3.2.

2.6 PROVISIONS FOR SEISMICALLY ISOLATED STRUCTURES:

2.6.1 GENERAL: Every seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this section and the applicable requirements of Chapter 1.

The lateral-force-resisting system and the isolation system shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions as provided in this section.

2.6.2 CRITERIA SELECTION:

2.6.2.1 Basis for Design: The procedures and limitations for the design of seismically isolated buildings shall be determined considering zoning, site characteristics, vertical acceleration, cracked section properties of concrete and masonry members, Seismic Hazard Exposure Group, configuration, structural system, and height in accordance with Sec. 2.2 except as noted below.

2.6.2.2 Stability of the Isolation System: The stability of the vertical load-carrying elements of the isolation system shall be verified by analysis and test, as required, for lateral seismic displacement equal to the total maximum displacement.

2.6.2.3 Seismic Hazard Exposure Group: All portions of the building, including the structure above the isolation system, shall be assigned a Seismic Hazard Exposure Group in accordance with the requirements of Sec. 1.4.3.

2.6.2.4 Configuration Requirements: Each building shall be designated as being regular or irregular on the basis of the structural configuration above the isolation system.

2.6.2.5 Selection of Lateral Response Procedure:

2.6.2.5.1 General: Any seismically isolated building may be and certain seismically isolated buildings defined below shall be designed using the dynamic lateral response procedure of Sec. 2.6.4.

2.6.2.5.2 Equivalent Lateral Force Procedure: The equivalent-lateral-response procedure of Sec. 2.6.3 may be used for design of a seismically isolated building provided that:

1. The building is located at least 15 km from all active faults;
2. The building is located on a Soil Profile Type A, B, C, or D site;
3. The structure above the isolation interface is less than or equal to four stories or 65 ft (20 m) in height;
4. The isolated period of the building, T_I, is less than or equal to 3.0 sec.
5. The isolated period of the building, T_I, is greater than three times the elastic, fixed-base period of the building above the isolation system as determined by Eq. 2.3.3.1-1 or 2.3.3.1-2;
6. The structure above the isolation system is of regular configuration; and
7. The isolation system meets all of the following criteria:
   a. The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement,
   b. The isolation system is capable of producing a restoring force as specified in Sec. 2.6.6.2.4,
   c. The isolation system has force-deflection properties that are independent of the rate of loading,
   d. The isolation system has force-deflection properties that are independent of vertical load and bilateral load, and
   e. The isolation system does not limit maximum capable earthquake displacement to less than M_M times the total design displacement.

2.6.2.5.3 Dynamic Analysis: A dynamic analysis may be used for the design of any building but shall be used for the design of all isolated buildings not satisfying Sec. 2.6.2.5.2. The dynamic lateral response procedure of Sec. 2.6.4 shall be used for design of seismically isolated buildings as specified below.

2.6.2.5.3.1 Response-Spectrum Analysis: Response-spectrum analysis may be used for design of a seismically isolated building provided that:

1. The building is located on a Soil Profile Type A, B, C, D, or E site and
2. The isolation system meets the criteria of Item 7 of Sec. 2.6.2.5.2.

2.6.2.5.3.2 Time-History Analysis: Time-history analysis may be used for design of any seismically isolated building and shall be used for design of all seismically isolated buildings not meeting the criteria of Sec. 2.6.2.5.3.1.

2.6.2.5.3.3 Site-Specific Design Spectra: Site-specific ground-motion spectra of the design earthquake and the maximum capable earthquake developed in accordance with Sec. 2.6.4.4.1 shall be used for design and analysis of all seismically isolated buildings if any one of the following conditions apply:

1. The building is located on a Soil Profile Type E or F site or

2. The building is located within 15 km of an active fault or

3. The isolated period of the building, T_I, is greater than 3.0 sec.

2.6.3 EQUIVALENT LATERAL FORCE PROCEDURE:

2.6.3.1 General: Except as provided in Sec. 2.6.4, every seismically isolated building or portion thereof shall be designed and constructed to resist minimum earthquake displacements and forces as specified by this section and the applicable requirements of Sec. 2.3.

2.6.3.2 Deformation Characteristics of the Isolation System: Minimum lateral earthquake design displacements and forces on seismically isolated buildings shall be based on the deformation characteristics of the isolation system.

The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used to meet the design requirements of this document.

The deformation characteristics of the isolation system shall be based on properly substantiated tests performed in accordance with Sec. 2.6.9.

2.6.3.3 Minimum Lateral Displacements:

2.6.3.3.1 Design Displacement: The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the building in accordance with the following:

$$D = \frac{g A_v F_v N_s T_I}{4\pi^2 B_I} \qquad (2.6.3.3.1)$$

where

g = acceleration of gravity. The units of the acceleration of gravity, g, are in./sec^2 (mm/sec^2) if the units of the design displacement, D, are inches (mm).

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

F_v = the soil coefficient as determined from Table 1.4.2.3b; for the calculation of D, use a value of $A_v = A_v N_s$ in Table 1.4.2.3b.

N_s = numerical coefficient related to both the proximity of the building to an active fault and fault magnitude as set forth in Table 2.6.3.3.1a.

T_I = period of seismically isolated building, in seconds (sec), in the direction under consideration, as prescribed by Eq. 2.6.3.3.2.

B_I = numerical coefficient related to the effective damping of the isolation system as set forth in Table 2.6.3.3.1b.

TABLE 2.6.3.3.1a
Near-Field Site Response Coefficient, N_s

Closest Distance, d_F, to Active Fault	Maximum Capable Earthquake Magnitude, M_{MCE}, of Active Fault[a,b]		
	$M_{MCE} \geq 8.0$	$M_{MCE} = 7.0$	$M_{MCE} \leq 6.0$
$d_F \geq 15$ km	1.0	1.0	1.0
$d_F = 10$ km	1.2	1.0	1.0
$d_F \leq 5$ km	1.5	1.3	1.1

[a] Location and maximum capable earthquake magnitude, M_{MCE}, of active faults shall be established from properly substantiated geotechnical data (e.g., most recent mapping of active faults by the U.S. Geological Survey).

[b] The near-field site response coefficient shall be based on linear interpolation of values of the closest distance, d_F, and maximum capable earthquake magnitude, M_{MCE}, other than those given.

TABLE 2.6.3.3.1b
Damping Coefficient, B_I

Effective Damping, β_I (Percentage of Critical)[a,b]	B_I Factor
≤ 2%	0.8
5%	1.0
10%	1.2
20%	1.5
30%	1.7
40%	1.9
≥ 50%	2.0

[a] The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Sec. 2.6.9.5.2.

[b] The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

2.6.3.3.2 Isolated-Building Period: The isolated-building period, T_I, shall be determined using the deformational characteristics of the isolation system in accordance with the following equation:

$$T_I = 2\pi \sqrt{\frac{W}{k_{min} g}} \qquad (2.6.3.3.2)$$

where:

W = total seismic dead load weight of the building above the isolation interface.

k_{min} = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

g = acceleration due to gravity.

2.6.3.3.3 Total Design Displacement: The total design displacement, D_T, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity.

The total design displacement, D_T, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the following equation:

$$D_T = \left[1 + y_p \frac{12 e_p}{b_p^2 + d_p^2}\right] \qquad (2.6.3.3.3)$$

where:

D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.

y_p = the distance, in feet (mm), between the center of rigidity of the isolation system rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration.

e_p = the actual eccentricity, in feet (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet (mm), taken as 5 percent of the maximum building dimension perpendicular to the direction of force under consideration.

b_p = the shortest plan dimension of the structure, in feet (mm), measured perpendicular to d_p.

d_p = the longest plan dimension of the structure, in feet (mm).

The total design displacement, D_T, may be taken as less than the value prescribed by Eq. 2.6.3.3.3, but not less than 1.1 times D, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

2.6.3.3.4 Total Maximum Displacement: The total maximum displacement, D_{TM}, required for verification of isolation system stability in the most critical direction of horizontal response shall be calculated in accordance with the following:

$$D_{TM} = M_M D_{TC} \qquad (2.6.3.3.4\text{-}1)$$

$$D_{TC} = D_C\left[1 + y_p \frac{12 e_p}{b_p^2 + d_p^2}\right] \qquad (2.6.3.3.4\text{-}2)$$

$$D_C = \frac{g A_v F_v N_s M_M T_I}{A \pi^2 B_I} \qquad (2.6.3.3.4\text{-}3)$$

where:

M_M = numerical coefficient related to maximum capable earthquake response as set forth in Table 2.6.3.3.4.

D_{TC} = the total design displacement for the maximum capable earthquake in inches (mm) including both translational displacement at the center of rigidity, D_c, and the component of torsional displacement in the direction under consideration.

D_C = the design displacement in inches (mm) at the center of rigidity of the isolation system for the maximum credible earthquake in the direction under consideration and F_v is based on a value of $A_v = M_M A_v N_s$ in Table 1.4.2.3b.

TABLE 2.6.3.3.4
Maximum Capable Earthquake Displacement Coefficient, M_M

Effective Peak Velocity-Related Acceleration, A_v	Response Region[a,b]	
	Constant Acceleration	Constant Velocity
0.40	1.2	1.25
0.30	1.33	1.5
0.20	1.5	1.75
0.15	1.67	2.0
0.10	2.0	2.5
0.05	2.5	3.0

[a] Except for construction of design spectra, as required in Sec. 2.6.4.4.1, the value of M_M shall be that specified for constant velocity.

[b] For construction of design spectra, as required in Sec. 2.6.4.4.1, the value of M_M specified for constant acceleration shall apply to all periods from 0 seconds to the transition period (i.e., 0.4 sec for Soil Profile Types A and B, 0.6 seconds for Soil Profile Types C and D, and 0.9 seconds for Soil Profile Type E), and the value of M_M specified for constant velocity shall apply to all periods greater than the transition period. The product $M_M A_v N$ in the constant velocity region need not exceed the product $M_M A_v N$ at the transition period.

2.6.3.4 Minimum Lateral Forces:

2.6.3.4.1 Isolation System and Structural Elements At or Below the Isolation System: The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the appropriate provisions for a nonisolated building where:

$$V_b = k_{max} D \qquad (2.6.3.4.1)$$

where:

V_b = the minimum lateral seismic design force or shear on elements of the isolation system or elements below the isolation system as prescribed by Eq. 2.6.3.4.1.

k_{max} = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.

2.6.3.4.2 Structural Elements Above the Isolation System: The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate provisions for a nonisolated building where:

$$V_s = \frac{k_{max} D}{R_I} \qquad (2.6.3.4.2)$$

where:

k_{max} = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.

R_I = numerical coefficient related to the type of lateral-force-resisting system above the isolation system.

The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system and shall be 3/8 of the R value given in Table 2.2.2 with an upper bound value not to exceed 2.0 and a lower bound value not to be less than 1.0.

2.6.3.4.3 Limits on V_s: The value of V_s shall not be taken as less than the following:

1. The lateral seismic force required by Sec. 2.3 for a fixed-base building of the same weight, W, and a period equal to the isolated period, T_I;

2. The base shear corresponding to the factored design wind load; and

3. The product of 1.5 times the lateral seismic force required to fully activate the isolation system (e.g., 1.5 times the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the static friction level of a sliding system).

2.6.3.5 Vertical Distribution of Force: The total force shall be distributed over the height of the structure above the isolation interface in accordance with the following equation:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{2.6.3.5}$$

where:

V_s = total lateral seismic design force or shear on elements above the isolation system as prescribed by Eq. 2.6.3.4.2.

W_x = portion of W that is located at or assigned to Level i, n, or x, respectively.

h_x = height above the base Level i, n, or x, respectively.

w_i = portion of W that is located at or assigned to Level i, n, or x, respectively.

h_i = height above the base Level i, n, or x, respectively.

At each level designated as x, the force, F_x, shall be applied over the area of the building in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

2.6.3.6 Drift Limits: The maximum interstory drift of the structure above the isolation system shall not exceed $0.010h_{sx}$. The drift shall be calculated by Eq. 2.3.7.1 with the C_d factor of the isolated structure equal to the R_I factor defined in Sec. 2.6.3.4.2.

2.6.4 DYNAMIC LATERAL RESPONSE PROCEDURE:

2.6.4.1 General: As required by Sec. 2.6.2, every seismically isolated building or portion thereof shall be designed and constructed to resist earthquake displacements and forces as specified in this section and the applicable requirements of Sec. 2.4.

2.6.4.2 Isolation System and Structural Elements Below the Isolation System: The total design displacement of the isolation system shall not be taken as less than 90 percent of D_T as specified by Sec. 2.6.3.3.3.

The total maximum displacement of the isolation system shall not be taken as less than 80 percent of D_{TM} as prescribed by Eq. 2.6.3.3.4.

The design lateral shear force on the isolation system and structural elements below the isolation system shall not be taken as less than 90 percent of V_b as prescribed by Eq. 2.6.3.4.1.

The limits of the first and third paragraphs of Sec. 2.6.4.2 shall be evaluated using values of D_T and D_{TM} determined in accordance with Sec. 2.6.3.3 except that D' may be used in lieu of D where D' is prescribed by the equation:

$$D' = \frac{D}{\sqrt{1 + \left(\frac{T}{T_I}\right)^2}} \tag{2.6.4.2}$$

where:

D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.

T = elastic, fixed-base period of the structure above the isolation system as determined by Sec. 2.3.3.

T_I = period of seismically isolated building, in seconds (sec), in the direction under consideration as prescribed by Eq. 2.6.3.3.2.

2.6.4.3 Structural Elements Above the Isolation System: The design lateral shear force on the structure above the isolation system, if regular in configuration, shall not be taken as less than 80 percent of $k_{max}D/R_I$ or less than the limits specified by Sec. 2.6.3.4.3.

> **EXCEPTION:** The design lateral shear force on the structure above the isolation system, if regular in configuration, may be taken as less than 80 percent $k_{max}D/R_I$, but not less than 60 percent of $k_{max}D/R_I$, provided time-history analysis is used for design of the structure.

The design lateral shear force on the structure above the isolation system, if irregular in configuration, shall not be taken as less than $k_{max}D/R_I$ or less than the limits specified by Sec. 2.6.3.4.3.

> **EXCEPTION:** The design lateral shear force on the structure above the isolation system, if irregular in configuration, may be taken as less than $k_{max}D/R_I$, but not less than 80 percent of $k_{max}D/R_I$, provided time-history analysis is used for design of the structure.

2.6.4.4 Ground Motion:

2.6.4.4.1 Design Spectra: Properly substantiated site-specific spectra are required for design of all buildings with an isolated period, T_I, greater than 3.0 sec or located on a Soil Profile Type E or F site or located within 15 km of an active fault. Buildings that do not require site-specific spectra and for which site-specific spectra have not been calculated shall be designed using spectra developed using the procedure presented in Table 2.6.4.4.1.

A design spectrum shall be constructed for the design earthquake. This design spectrum shall not be taken as less than the response spectrum developed using Table 2.6.4.4.1 for the appropriate Soil Profile Type.

EXCEPTION: If a site-specific spectrum is calculated for the design earthquake, the design spectrum may be taken as less than 100 percent but not less than 80 percent of the response spectrum developed using Table 2.6.4.4.1 for the appropriate Soil Profile Type.

A design spectrum shall be constructed for the maximum capable earthquake. This design spectrum shall not be taken as less than the spectrum developed using Table 2.6.4.4.1 for the appropriate Soil Profile Type. This design spectrum shall be used to determine the total maximum displacement and overturning forces for design and testing of the isolation system.

EXCEPTION: If a site-specific spectrum is calculated for the maximum capable earthquake, the design spectrum may be taken as less than 100 percent but not less than 80 percent of the response spectrum developed using Table 2.6.4.4.1 for the appropriate Soil Profile Type.

TABLE 2.6.4.4.1
Construction of Response Spectra (free field, elastic, smoothed, 5 percent damping)

Step 1	Select A_a and A_v from Maps 3 and 4.
Step 2	Select the appropriate Soil Profile Type from Sec. 1.4.2.
Step 3	Determine the corresponding values of the soil coefficients F_a and F_v and F_a' and F_v' from Tables 1.4.2.3a and 1.4.2.3b. The values of F_a and F_v shall be determined using a value of $A_v = A_v M_s$ for the design spectra. The values of F_a' and F_v' shall be determined using a value of $A_v = M_M A_s N_s$ for the maximum capable response spectra.
Step 4	Compute the (short period) constant spectral acceleration portion of the response as follows: Design spectra $S_A = 2.5 F_a A_a N_s$ (2.6.4.4.1-1) Maximum capable spectra $S_A = 2.5 M_M F_a' A_a N_s$ (2.6.4.4.1-2)
Step 5	Compute the (longer period) constant velocity portion of the spectrum (S_A decreases as $1/T$ where T = period) as follows: Design spectra $S_A = F_v A_v N_s \left(\frac{1}{T}\right)$ (2.6.4.4.1-3)
Step 6	At each period (T), the elastic response spectrum is the lesser of the two values from Eq. 2.6.4.4.1-1 and 2.6.4.4.1-3 so that Eq. 2.6.4.4.1-1 defines S_A in the low-period range and Eq. 2.6.4.4.1-3 defines S_A in the higher period range. The period at which the transition from Eq. 2.6.4.4.1-1 to Eq. 2.6.4.4.1-3 occurs varies as a function of: (a) the Soil Profile Type, (b) A_a, and (c) A_v.

2.6.4.4.2 Time Histories: Pairs of horizontal ground-motion time-history components shall be selected from not less than three recorded events. For each pair of horizontal ground-motion components, the square root sum of the squares (SRSS) of the 5 percent damped spectrum of the scaled, horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.3 times the 5 percent damped spectrum of the design earthquake (or maximum capable earthquake) by more than 10 percent in the period range of T_I, as determined by Eq. 2.6.3.3.2, for periods from T_I minus 1.0 sec to T_I plus 1.0 sec.

The duration of the time histories shall be consistent with the magnitude and source characteristics of the design earthquake (or maximum capable earthquake).

Time histories developed for sites within 15 km of a major active fault shall incorporate near-fault phenomena.

2.6.4.5 Mathematical Model:

2.6.4.5.1 General: The mathematical models of the isolated building including the isolation system, the lateral-force-resisting system, and other structural elements shall conform to Sec. 2.4.2 and to the requirements of Sec. 2.6.4.5.2 and 2.6.4.5.3, below.

2.6.4.5.2 Isolation System: The isolation system shall be modeled using deformational characteristics developed and verified by test in accordance with the requirements of Sec. 2.6.3.2. The isolation system shall be modeled with sufficient detail to:

1. Account for the spatial distribution of isolator units;

2. Calculate translation, in both horizontal directions, and torsion of the building above the isolation interface considering the most disadvantageous location of mass eccentricity;

3. Assess overturning/uplift forces on individual isolator units; and

4. Account for the effects of vertical load, bilateral load, and/or the rate of loading if the force deflection properties of the isolation system are dependent on one or more of these attributes.

2.6.4.5.3 Isolated Building:

2.6.4.5.3.1 Displacement: The maximum displacement of each floor and the total design displacement and total maximum displacement across the isolation system shall be calculated using a model of the isolated building that incorporates the force-deflection characteristics of nonlinear elements of the isolation system and the lateral-force-resisting system.

Isolation systems with nonlinear elements include, but are not limited to, systems that do not meet the criteria of Item 7 of Sec. 2.6.2.5.2.

Lateral-force-resisting systems with nonlinear elements include, but are not limited to, irregular structural systems designed for a lateral force less than $k_{max}D/R_I$ and regular structural systems designed for a lateral force less than 80 percent of $k_{max}D/R_I$.

2.6.4.5.3.2 Forces and Displacements in Key Elements: Design forces and displacements in key elements of the lateral force-resisting system may be calculated using a linear elastic model of the isolated structure provided that:

1. Pseudo-elastic properties assumed for nonlinear isolation-system components are based on the maximum effective stiffness of the isolation system and

2. All key elements of the lateral-force-resisting system are linear.

2.6.4.6 Description of Analysis Procedures:

2.6.4.6.1 General: Response-spectrum and time-history analyses shall be performed in accordance with Sec. 2.4 and the requirements of this section.

2.6.4.6.2 Input Earthquake: The design earthquake shall be used to calculate the total design displacement of the isolation system and the lateral forces and displacements of the isolated structure. The maximum capable earthquake shall be used to calculate the total maximum displacement of the isolation system.

2.6.4.6.3 Response-Spectrum Analysis. Response-spectrum analysis shall be performed using a damping value equal to the effective damping of the isolation system or 30 percent of critical, whichever is less.

Response-spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the most critical direction of ground motion and 30 percent of the ground motion on the orthogonal axis. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

The design shear at any story shall not be less than the story shear obtained using Eq. 2.6.3.5 and a value of V_s taken as that equal to the base shear obtained from the response-spectrum analysis in the direction of interest.

2.6.4.6.4 Time-History Analysis: Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components as defined in Sec. 2.6.4.4.2.

Each pair of time histories shall be applied simultaneously to the model considering the most disadvantageous location of mass eccentricity. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal components at each time step.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, the average value of the response parameter of interest shall be used for design.

2.6.4.7 Design Lateral Force

2.6.4.7.1 Isolation System and Structural Elements At or Below the Isolation System: The isolation system, foundation, and all structural elements below the isolation system shall be designed using all of the appropriate provisions for a nonisolated building and the forces obtained from the dynamic analysis without reduction.

2.6.4.7.2 Structural Elements Above the Isolation System: Structural elements above the isolation system shall be designed using the appropriate provisions for a nonisolated building and the forces obtained from the dynamic analysis reduced by a factor of R_I. The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system.

2.6.4.7.3 Scaling of Results: When the factored lateral shear force on structural elements, determined using either response spectrum or time-history analysis, is less than the minimum level prescribed by Sec. 2.6.4.2 and 2.6.4.3, all response parameters, including member forces and moments, shall be adjusted upward proportionally.

2.6.4.7.4 Drift Limits: Maximum interstory drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum interstory drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed $0.015h_{sx}$ and

2. The maximum interstory drift of the structure above the isolation system calculated by time-history analysis considering the force-deflection characteristics of nonlinear elements of the lateral-force-resisting system shall not exceed $0.020h_{sx}$.

Drift shall be calculated using Eq. 2.3.7.1 with the C_d factor of the isolated structure equal to the R_I factor defined in Sec. 2.6.3.4.2.

The secondary effects of the maximum capable earthquake lateral displacement Δ of the structure above the isolation system combined with gravity forces shall be investigated if the interstory drift ratio exceeds $0.010/R_I$.

2.6.5 LATERAL LOAD ON ELEMENTS OF BUILDINGS AND NONSTRUCTURAL COMPONENTS SUPPORTED BY BUILDINGS:

2.6.5.1 General: Parts or portions of an isolated building, permanent nonstructural components and the attachments to them, and the attachments for permanent equipment supported by a building shall be designed to resist seismic forces and displacements as prescribed by this section and the applicable requirements of Chapter 3.

2.6.5.2 Forces and Displacements:

2.6.5.2.1 Components At or Above the Isolation Interface: Elements of seismically isolated buildings and nonstructural components, or portions thereof, that are at or above the isolation interface shall be designed to resist a total lateral seismic force equal to the maximum dynamic response of the element or component under consideration.

EXCEPTION: Elements of seismically isolated buildings and nonstructural components or portions thereof may be designed to resist total lateral seismic force as prescribed by Eq. 2.2.6-1 or 2.2.6-2 as appropriate.

2.6.5.2.2 Components Crossing the Isolation Interface: Elements of seismically isolated buildings and nonstructural components, or portions thereof, that cross the isolation interface shall be designed to withstand the total maximum displacement.

2.6.5.2.3 Components Below the Isolation Interface: Elements of seismically isolated buildings and nonstructural components, or portions thereof, that are below the isolation interface shall be designed and constructed in accordance with the requirements of Sec. 2.2.

2.6.6 DETAILED SYSTEM REQUIREMENTS:

2.6.6.1 General: The isolation system and the structural system shall comply with the material requirements of Chapters 5 through 9. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Sec. 2.2.

2.6.6.2 Isolation System:

2.6.6.2.1 Environmental Conditions: In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

2.6.6.2.2 Wind Forces: Isolated buildings shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

2.6.6.2.3 Fire Resistance: Fire resistance for the isolation system shall meet that required for the building columns, walls, or other structural elements.

2.6.6.2.4 Lateral-Restoring Force: The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

EXCEPTION: The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement or $36M_M C_a N_s$ inches (or $915M_M C_a N_s$ mm).

2.6.6.2.5 Displacement Restraint: The isolation system may be configured to include a displacement restraint that limits lateral displacement due to the maximum capable earthquake to less than M_M times the total design displacement provided that the seismically isolated building is designed in accordance with the following criteria when more stringent than the requirements of Sec. 2.6.2:

1. Maximum capable earthquake response is calculated in accordance with the dynamic analysis requirements of Sec. 2.6.4 explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.

2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum capable earthquake.

3. The structure above the isolation system is checked for stability and ductility demand of the maximum capable earthquake, and

4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

2.6.6.2.6 Vertical-Load Stability: Each element of the isolation system shall be designed to be stable under the full-design-vertical load at a horizontal displacement equal to the total maximum displacement. Full-design-vertical load shall be computed using maximum and minimum vertical loads based on the combination of factored gravity and seismic loads as specified in Sec. 2.2.6. The seismic load E is given by Eq. 2.2.6-1 and 2.2.6-2 where C_a in these equations is replaced by $M_M N_s C_a$. The vertical load due to earthquake, Q_E, shall be based on peak response due to the maximum capable earthquake.

2.6.6.2.7 Overturning: The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum capable earthquake and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the isolator units or other building elements.

2.6.6.2.8 Inspection and Replacement: Access for inspection and replacement of all components of the isolation system shall be provided.

2.6.6.2.9 Quality Control: A quality control testing program for isolator units shall be established by the engineer responsible for the structural design.

2.6.6.3 Structural System:

2.6.6.3.1 Horizontal Distribution of Force: A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another.

2.6.6.3.2 Building Separations: Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

2.6.6.3.3 Nonbuilding Structures: These shall be designed and constructed in accordance with the requirements of Sec. 2.7 using design displacements and forces calculated in accordance with Sec. 2.6.3 or 2.6.4.

2.6.7 FOUNDATIONS: Foundations shall be designed and constructed in accordance with the requirements of Chapter 4 using design forces calculated in accordance with Sec. 2.6.3 or 2.6.4, as appropriate.

2.6.8 DESIGN AND CONSTRUCTION REVIEW:

2.6.8.1 General: A design review of the isolation system and related test programs shall be performed by an independent engineering team including persons licensed in the appropriate disciplines and experienced in seismic analysis methods and the theory and application of seismic isolation.

2.6.8.2 Isolation System: Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria including the development of site-specific spectra and ground motion time histories and all other design criteria developed specifically for the project;

2. Review of the preliminary design including the determination of the total design displacement of the isolation system design displacement and the lateral force design level;

3. Overview and observation of prototype testing (Sec. 2.6.9);

4. Review of the final design of the entire structural system and all supporting analyses; and

5. Review of the isolation system quality control testing program (Sec. 2.6.6.2.9).

2.6.9 REQUIRED TESTS OF THE ISOLATION SYSTEM:

2.6.9.1 General: The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated buildings shall be based on tests of a selected sample of the components prior to construction as described in this section.

The isolation system components to be tested shall include the wind-restraint system if such a system is used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests of Sec. 2.6.6.2.9.

2.6.9.2 Prototype Tests

2.6.9.2.1 General: Prototype tests shall be performed separately on two full-size specimens of each predominant type and size of isolator unit of the isolation system. The test specimens shall include the wind restraint system as well as individual isolator units if such systems are used in the design. Specimens tested shall not be used for construction.

2.6.9.2.2 Record: For each cycle of tests, the force-deflection and hysteretic behavior of the test specimen shall be recorded.

2.6.9.2.3 Sequence and Cycles: The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force;

2. Three fully reversed cycles of loading at each of the following increments of the total design displacement--0.25, 0.50, 0.75, and 1.0;

3. Three fully reversed cycles of loading at the total maximum displacement; and

4. $15F_V/B_I$, but not less than ten, fully reversed cycles of loading at 1.0 times the total design displacement.

If an isolator unit is also a vertical-load-carrying element, then Item 2 of the sequence of cyclic tests specified above shall be performed for two additional vertical load cases. The first is given by the combination of the average dead load plus half the live load plus the earthquake load from Eq. 2.2.6-1. The second is given by the average dead load minus the earthquake load from Eq. 2.2.6-2. In Eq. 2.2.6-1 and 2.2.6-2 where C_a is replaced by C_aN_s and where the vertical load due to earthquake, Q_E, shall be based on peak response due to the design earthquake. In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size.

2.6.9.2.4 Units Dependent on Loading Rates: If the force-deflection properties of the isolator units are dependent on the rate of loading, each set of tests specified in Sec. 2.6.9.2.3 shall be performed at a frequency, f, in the range of 0.1 to 1.0 times the inverse of the effective period, T_s. The frequency, f, shall be the minimum frequency of testing at which the effective stiffness and the effective damping at the design displacement are at least 85 percent of the corresponding values when the isolator unit is tested at a frequency equal to the inverse of period T_s.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 15 percent difference in the effective stiffness and the effective damping at the design displacement when tested at a frequency equal to the inverse of the effective period of the isolated building and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period of the isolated building.

2.6.9.2.5 Units Dependent on Bilateral Load: If the force-deflection properties of the isolator units are dependent on bilateral load, the tests specified in Sec. 2.6.9.2.3 and 2.6.9.2.4 shall be augmented to include bilateral load at the following increments of the total design displacement: 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0.

If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, the reduced scale specimens shall be of the same type and material and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load if the bilateral and unilateral force-deflection properties have greater than a 15 percent difference in effective stiffness at the design displacement.

2.6.9.2.6 Downward-Vertical Load: Isolator units that carry vertical load shall be statically tested for maximum and minimum downward vertical load at the total maximum displacement. In these tests, the maximum and minimum vertical load on any one isolator unit of a common type and size shall be based on the combination of factored gravity and seismic loads as specified in Sec. 2.2.6. The seismic load, E, is given by Eq. 2.2.6-1 and Eq. 2.2.6-2 where C_a in these equations is replaced by $M_M N_s C_a$ and the vertical load, Q_E, is based on the peak response of the maximum capable earthquake.

2.6.9.2.7 Sacrificial-Wind-Restraint Systems: If a sacrificial-wind-restraint system is to be utilized, the ultimate capacity shall be established by test.

2.6.9.2.8 Testing Similar Units: The prototype tests are not required if an isolator unit is of similar size and of the same type and material as a prototype isolator unit that has been previously tested using the specified sequence of tests.

2.6.9.3 Determination of Force-Deflection Characteristics: The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.

The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_I^+ - F_I^-}{\Delta_I^+ - \Delta_I^-} \qquad (2.6.9.3)$$

where F_I^+ and F_I^- are the maximum positive and maximum negative forces, respectively, and Δ_I^+ and Δ_I^- are the maximum positive and maximum negative test displacements, respectively.

If the minimum effective stiffness is to be determined, F^+_{min} and F^-_{min} shall be used in the equation.

If the maximum effective stiffness is to be determined, F^+_{max} and F^-_{max} shall be used in the equation.

2.6.9.4 System Adequacy: The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

1. For each increment of test displacement specified in Item 2 of Sec. 2.6.9.2.3 and for each vertical load case specified in Sec. 2.6.9.2.3:

   There is no greater than a 15 percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen;

2. For each increment of test displacement specified in Item 2 of Sec. 2.6.9.2.3 and for each vertical load case specified in Sec. 2.6.9.2.3;

   There is no greater than a 15 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test;

3. For each specimen there is no greater than a plus or minus 20 percent change in the initial effective stiffness of each test specimen over the $15F_v/B_I$, but not less than 10, cycles of test specified in Item 3 of Sec. 2.6.9.2.3;

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the $15F_v/B_I$, but not less than 10, cycles of test specified in Sec. 2.6.9.2.3; and

5. All specimens of vertical-load-carrying elements of the isolation system remain stable up to the total maximum displacement for static load as prescribed in Sec. 2.6.9.2.6 and shall have a positive incremental force-carrying capacity.

2.6.9.5 Design Properties of the Isolation System:

2.6.9.5.1 Effective Stiffness: The minimum and maximum effective stiffness of the isolation system shall be determined as follows:

1. The value of k_{min} shall be based on the minimum effective stiffness of individual isolator units as established by the cyclic tests of Item 2 of Sec. 2.6.9.2.3 at a displacement amplitude equal to the design displacement;

2. The value of k_{max} shall be based on the maximum effective stiffness of individual isolator units as established by the cyclic tests of Item 2 of Sec. 2.6.9.2.3 at a displacement amplitude equal to the design displacement; and

3. For isolator units that are found by the tests of Sec. 2.6.9.2.3, 2.6.9.2.4, or 2.6.9.2.5 to have force-deflection characteristics that vary with vertical load, rate of loading or bilateral load, respectively, the value of k_{max} shall be increased and the value of k_{min} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

2.6.9.5.2 Effective Damping: The effective damping, β_I, of the isolation system shall be calculated as follows:

$$\beta_I = \frac{\text{Total Area}}{2\pi k_{max} D^2} \tag{2.6.9.5.2}$$

where the total area shall be taken as the sum of the areas of the hysteresis loops of all isolator units and the hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of Item 2 of Sec. 2.6.9.2.3 at a displacement amplitude equal to the design displacement and

k_{max} = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.

2.7. PROVISIONS FOR NONBUILDING STRUCTURES:

2.7.1 GENERAL:

2.7.1.1: Nonbuilding structures include all self-supporting structures, other than buildings, bridges and dams, that are supported by the earth, that carry gravity loads, and that may be required to resist the effects of earthquake. Nonbuilding structures shall be designed to resist the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by this section.

2.7.1.2: The design of nonbuilding structures shall provide sufficient strength and ductility, consistent with the requirements specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces:

a. Applicable strength and other design criteria shall be obtained from other sections of the *Provisions* or its referenced codes and standards.

b. When applicable strength and other design criteria are not contained in or referenced by the *Provisions*, such criteria shall be obtained from approved national standards. Where approved national standards define acceptance criteria in terms of allowable stresses as opposed to strength, the design seismic forces shall be obtained from the *Provisions* and **reduced** by a factor of 1.5 for use with allowable stresses. Detailing shall be in accordance with the approved national standards.

2.7.1.3: The weight W for nonbuilding structures shall include all dead load as defined for buildings in Sec. 2.3.2. For purposes of calculating design seismic forces in nonbuilding structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, and piping.

2.7.1.4: The fundamental period of the nonbuilding structure shall be determined by rational methods as prescribed in Sec. 2.3.3.

2.7.1.5: The drift limitations of Sec. 2.2.7 need not apply to nonbuilding structures. Drift limitations shall be established for structural and nonstructural elements whose failure would cause life-safety hazards. P-delta effects shall be considered for structures whose drifts exceed one-half the values in Sec. 2.2.7.

2.7.1.6: At sites where the seismic coefficient C_a is greater than or equal to 0.20, nonbuilding structures that support flexible nonstructural elements whose combined gravity weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported element.

2.7.2: The lateral force procedure for nonbuilding structures with structural systems similar to buildings (those with structural systems listed in Table 2.2.2) shall be selected in accordance with the force and detailing requirements of Sec. 2.2.1.

> **EXCEPTION:** Intermediate moment frames of reinforced concrete may be used at sites where the seismic coefficient C_a is greater than or equal to 0.20 if:
>
> 1. The nonbuilding structure is less than 50 ft (15.2 m) in height and
>
> 2. $R = 3.0$ is used for design.

If hazardous contents are supported, the seismic base shear (V) shall be increased by 50 percent.

2.7.3: Nonbuilding structures that have a fundamental period, T, less than 0.06 sec, including their anchorages, shall be designed for the lateral force obtained from the following:

$$V = 0.60 C_a W \tag{2.7.3}$$

where:

V = seismic design force applied to a nonbuilding structure,

C_a = the seismic coefficient based upon the Soil Profile Type and the Value of A_a as determined from Sec. 1.4.2.3 or Table 1.4.2.4a,

W = nonbuilding structure operating weight.

The force shall be distributed with height in accordance with Sec. 2.3.4.

2.7.4: Flat-bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist seismic forces calculated using the procedures in Sec. 3.3.9.2.

2.7.5: Nonbuilding structures that are not covered by Sec. 2.7.2 through 2.7.4 shall be designed to resist minimum seismic lateral forces not less than those determined in accordance with the requirements of Sec. 2.3.2 with the following additions and exceptions:

1. The response modification coefficient, R, shall be as given in Table 2.7.5. The ratio of $\frac{1.2C_v}{RT^{2/3}}$ used in design shall not be less than $0.50C_a$.

TABLE 2.7.5
Response Modification Coefficients for Nonbuilding Structures

Nonbuilding Structure Type	R
Vessels (including tanks and pressurized spheres) on braced or unbraced legs Hazardous contents All others	 1.5 2
Cast-in-place concrete silos and chimneys having walls continuous to the foundation	3.5
All other distributed mass cantilever structures not covered above including stacks, chimneys, silos, and skirt-supported vertical vessels Hazardous contents All other	 2 3
Trussed towers (freestanding or guyed), guyed stacks and chimneys	3
Inverted pendulum type structures	2
Cooling towers	3.5
Bins and hoppers on braced or unbraced legs	3
Signs and billboards	3.5
Amusement structures and monuments	2
All other self-supporting structures not covered above	3

2. The vertical distribution of the lateral seismic forces in nonbuilding structures covered by this section may be determined:

   a. Using the requirements of Sec. 2.3.4 or

   b. Using the procedures of Sec. 2.4.

   Irregular structures at sites where the seismic coefficient C_a is greater than or equal to 0.20 that cannot be modelled as a single mass shall use the procedures of Sec. 2.4.

3. Where an approved national standard provides a basis for the earthquake resistant design of a particular type of nonbuilding structure covered by Sec. 2.7, such a standard may be used subject to the following limitations:

   a. The seismic ground acceleration and seismic coefficient shall be in conformance with the requirements of Sec. 1.4.1 and 1.4.2, respectively.

   b. The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the value that would be obtained using the *Provisions*.

Appendix to Chapter 2

PASSIVE ENERGY DISSIPATION SYSTEMS

PREFACE: The *NEHRP Recommended Provisions* is a resource document, not a model code; therefore, this appendix is included to introduce to potential users new and relevant techniques for incorporating energy dissipation devices into earthquake-resistant building designs and to provide the requirements necessary to ensure that the devices will perform as designed. An independent engineering panel should be used to review each building application of passive energy dissipation systems

The technical basis for Sec. 2A.1 through 2A.3 of this appendix is well established, and energy dissipation devices have been utilized in the seismic rehabilitation of buildings in Canada, Japan, Mexico, and the United States; however, the design requirements provided in this appendix have not been evaluated under typical design office conditions. The technical basis for Sec. 2A.4 has been discussed thoroughly by engineers, researchers, and some current energy dissipation device vendors; however, it is important that no specific device be excluded unnecessarily.

Review of, trial designs based on, and comments on this appendix are encouraged. Please direct such feedback to the BSSC.

2A.1 GENERAL: This appendix provides minimum design requirements for the incorporation of energy dissipation devices in buildings. In addition to the requirements provided herein, every structure incorporating energy dissipators and every portion thereof should be designed in accordance with the applicable provisions of Sec. 2.2, 2.3, 2.4, 2.5, and 2.6 unless modified by the requirements of this appendix. The energy dissipation system should be designed with consideration given to other environmental conditions including wind, aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances. Structures incorporating energy dissipation systems should provide access for inspection and replacement of the devices. Tests of energy dissipation devices should be made in accordance with Sec. 2A.4 to establish and validate their design properties prior to their installation.

2A.2 STRUCTURAL FRAMING SYSTEMS: Application of energy dissipation devices is limited to moment resisting frame systems and dual systems as defined in Sec. 2.2.2. To be effective, the energy dissipation devices must connect two portions of a structure that have relative motions during an earthquake. If it can be shown that relative motions are large enough to adequately mobilize the energy dissipation devices, other structural systems can be selected.

The building height limitations and the response modification values, R, should not exceed the values for the primary lateral load resisting system as provided in Table 2.2.2.

At least two energy dissipation devices should be provided at each story in each principal direction of the building framing. These devices should be provided continuously from the base to the top of the building unless it is demonstrated that satisfactory performance is achieved with an incomplete vertical distribution.

2A.3 ANALYSIS PROCEDURES: The minimum design forces should be no lower than the product of the minimum values of Sec. 2.3, 2.4, 2.5, or 2.6 and the values given in Table 2A.3. Structural members that transmit the forces from the energy dissipation devices to the foundation should be designed to remain elastic for 1.2 times the maximum device forces associated with the design level earthquake.

For Seismic Performance Category A and B buildings, the structural analysis should be made in accordance with Sec. 2.2.4.

For Seismic Performance Category C buildings, the procedures of Sec. 2A.3.1 for linear viscous systems or Sec. 2A.3.2.1 for other energy dissipation systems should be used in addition to the requirements of Sec. 2.2.4. If both linear viscous and other energy dissipation devices are used in a building, the requirements of Sec. 2A.3.2.1 apply.

For Seismic Performance Category D and E buildings, the nonlinear dynamic lateral response requirements of Sec. 2A.3.2.2 should be used in addition to the requirements of Sec. 2.2.4.

> **EXCEPTION:** Seismic Performance Category D and E buildings need satisfy only the requirements of Sec. 2.2.4 and 2A.3.1 when the energy dissipation system uses only linear viscous devices.

TABLE 2A.3
Reduction Factors for Increased Building Damping

Fraction of Critical Damping	Reduction Factor
0.05	1.00
0.10	0.84
0.15	0.72
0.20	0.64
0.25	0.58
0.30	0.53

2A.3.1 Linear Viscous Devices: These devices have a linear force versus relative velocity of motion between their ends as specified in Sec. 2A.4. The seismic base shear force for buildings designed using linear viscous devices should be obtained by multiplying the seismic base shear determined from Eq. 2.3.2 by the damping reduction factor given in Table 2A.3. These tabular values are for the modal total fraction of critical damping (inherent plus added by the viscous devices). Either the larger value or linear interpolation between tabular values may be used. The reduced forces should be used for the design of the lateral load resisting system of the building and the determination of deflection limits as provided in Sec. 2.3

Damping beyond 30 percent of critical requires nonlinear time history analyses as described in Sec. 2A.3.2.

2A.3.2 Other Energy Dissipation Devices: For energy dissipation systems incorporating devices other than the linear viscous devices covered in Sec. 2A.3.1, the lateral-force-resisting system (including the energy dissipation system) should satisfy the requirements of this section. A two-step design and evaluation process is required for all Seismic Performance Category D and E buildings using energy dissipation devices other than linear viscous devices covered in Sec. 2A.3.1. The first step treats the system as an equivalent viscously damped system for preliminary design. The second step requires nonlinear time history analyses of the building system to verify the expected performance.

2A.3.2.1 Equivalent Viscously Damped System: The effective stiffness and the effective damping of the energy dissipation system should be determined from the device tests performed in accordance with Sec. 2A.4 and from analyses of the device supporting system.

The effective stiffness of the energy dissipation system should be taken as the average of the absolute values of the positive and negative forces recorded at the corresponding design displacements divided by the design displacement. If the tests include only the device, the device stiffness should be combined with the device supporting system to calculate the effective stiffness of the dissipation system. This effective energy dissipation system stiffness should be combined with the lateral resisting framing stiffnesses to establish the lateral stiffnesses of the building.

The effective device damping coefficient c_{eq} should be taken as:

$$c_{eq} = \frac{W_D T}{2\pi^2 \Delta^2} \qquad (2A.3.2.1)$$

where:

W_D = the energy dissipated per cycle at the story displacement for the design earthquake, which is equal to the area of the hysteresis curve determined in accordance with Sec. 2A.4;

T = the modal period of the building modified as appropriate to account for the effective stiffness of the energy dissipation system; and

Δ = the corresponding displacement of the device and device supports across the story.

The increase in effective modal damping of the building system should be calculated using computed c_{eq} values distributed appropriately throughout the building. The modal approach provided in Sec. 2.4 should be used where the base shear in each mode is multiplied by the reduction factor of Table 2A.3 as determined from the effective modal damping values.

2A.3.2.2 Nonlinear Dynamic Response Verification: The building design resulting from application of the provisions of Sec. 2A.3.2.1 should be verified by a nonlinear analysis of the building as described below. The curvature and displacement ductility demands on the structural members should be demonstrated by moment-curvature analysis to be adequate and interstory displacements should be less than those specified in Table 2.2.7. The analytical device performance should be accurately modeled to represent the test results obtained in accordance with Sec. 2A.4.

2A.3.2.2.1 Ground Motion Records: For two-dimensional nonlinear analyses, three horizontal ground motion accelerograms should be selected. Each motion should be scaled such that the ordinates of its 5 percent damped spectrum in the period range of T - 0.5 sec to T + 0.5 sec should not be less than the design spectrum specified in Sec. 2.4 but not reduced by R where T includes the period of all modes that contribute at least 5 percent of the total modal mass.

For three-dimensional nonlinear analyses, three pairs of horizontal ground motion records should be selected and scaled as specified in Sec. 2.6.4.4.

The duration of the ground motion time history records should be consistent with the magnitude and source characteristics of the design earthquake for the site. Ground motion time histories for sites within 15 km (9.3 mi) of a major active fault should incorporate near-fault phenomena.

2A.3.2.2.2 Mathematical Model: The three-dimensional mathematical models used for the analysis of a building including the energy dissipation system should meet the requirements of Sec. 2.4. The energy dissipating devices should be modeled as discrete elements in a detailed structural frame. The model should account for both the plan and vertical spatial distribution of the energy dissipation devices throughout the building and the dependence, if any, of the device characteristics on loading frequency, temperature, sustained loads, nonlinearities, and bilateral loads. The energy dissipation system should be modeled in sufficient detail to account for translational and torsional responses of the building considering the most disadvantageous location of the mass eccentricity.

> **EXCEPTION:** If both the lateral-load-resisting system of the building and the energy dissipation system satisfy the plan regularity requirements of Sec. 2.2.3, two-dimensional mathematical models may be used in lieu of three-dimensional models.

2A.4 TESTING:

2A.4.1 Prototype Tests: Prototype tests should be performed separately on at least two full-size energy dissipation devices of each predominant type and size to be used in the construction. For the following test sequence, the device should be loaded with a vertical load equal to the average dead load on the device as installed in the building and each cycle of the hysteretic behavior of the device should be recorded:

a. 200 fully reversed cycles of lateral force corresponding to the wind design force,

b. 50 fully reversed cycles at the device displacement corresponding to the design level earthquake, and

c. 10 fully reversed cycles at M_M times the device displacement corresponding to the design level earthquake where M_M is as given in Table 2.6.3.3.4.

If the force-deformation properties of the energy dissipation devices change more than 10 percent for changes in testing frequency to half or twice the fundamental frequency of the building, the preceding tests should be performed at 0.5, 1, and 2 times the fundamental frequency of the building. If the devices are subjected to bilateral load or if their force-deformation properties are dependent on bilateral load, the preceding tests should be made at both zero and full design level bilateral displacement.

> **EXCEPTION:** Energy dissipation devices of a similar size and material composition that have been tested previously in the manner described above need not be tested provided that all of the mechanical characteristics of the devices can be similitude scaled and the data are made available to the owner.

2A.4.2 Force-Deformation Characteristics: The force-deformation characteristics of an energy dissipation device should be based on the cyclic test results of Sec. 2A.4.1. The device equivalent viscous damping should be determined from Eq. 2A.3.2.1 where W_D is the area of the hysteresis curve at the design displacements. The device equivalent stiffness should be determined as provided in Sec. 2A.3.2.1.

2A.4.3 System Adequacy: The performance of the prototype energy dissipation device should be assessed as adequate if the following conditions are satisfied:

a. The force-deformation hysteresis curves for the tests of Sec. 2A.4.1 have non-negative incremental force-carrying capacities.

   > **EXCEPTION:** Energy dissipation devices that exhibit frequency-dependent hysteretic behavior need not comply with this requirement.

b. Within each test sequence of Sec. 2A.4.1, the effective stiffness of a prototype energy dissipation device for any one cycle does not differ by more than 15 percent from the average effective stiffness as calculated from all tests of that sequence.

   > **EXCEPTION:** Energy dissipation devices that exhibit frequency-dependent or temperature-dependent hysteretic behavior need not comply with this requirement provided that it is demonstrated by analysis that the variation in effective stiffness measured during testing does not have a deleterious effect on the response of the building.

c. Within each test sequence of Sec. 2A.4.1, the area of the hysteresis curves for a prototype energy dissipation device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis curve as calculated from all tests of that sequence.

   > **EXCEPTION:** Energy dissipation devices that exhibit frequency-dependent or temperature-dependent hysteric behavior need not comply with this requirement provided that it is demonstrated by analysis that the variation in the area of the hysteresis curves (equivalent viscous damping values) measured during testing does not have a deleterious effect on the response of the building.

Chapter 3

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS

3.1 GENERAL: This chapter establishes minimum design criteria for architectural, mechanical, electrical, and nonstructural systems, components, and elements permanently attached to buildings, including supporting structures and attachments (hereinafter referred to as "components"). The design criteria establish minimum equivalent static force levels and relative displacement demands for the design of components and their attachments to the structure, recognizing ground motion and structural amplification, component toughness and weight, and performance expectations.

Seismic Performance Categories for buildings are defined in Sec. 1.4.4. For the purposes of this chapter, components shall be considered to have the same Seismic Performance Category as that of the building that they occupy or to which they are attached unless otherwise noted.

In addition, all components are assigned a component importance factor (I_p) in this chapter. The default value for I_p is 1.00 for typical components in normal service. Higher values for I_p are assigned for components which contain hazardous substances, must have a higher level of assurance of function, or otherwise require additional attention because of their life-safety characteristics. Component importance factors are prescribed in Sec. 3.1.5.

All architectural, mechanical, electrical, and other nonstructural components in buildings shall be designed and constructed to resist the equivalent static forces and displacements determined in accordance with this chapter. The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength.

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

1. All components in Seismic Performance Category A,
2. Architectural components in Seismic Performance Category B when the importance factor (I_p) is equal to 1.00,
3. Mechanical and electrical components in Seismic Performance Categories B and C when the importance factor (I_p) is equal to 1.00, or
4. Mechanical and electrical components in all Seismic Performance Categories that are mounted at 4 ft (1.22 m) or less above a floor level <u>and</u> weigh 400 lb (1780 N) or less, when the importance factor (I_p) is equal to 1.00.

The interrelationship of components and their effect on each other shall be considered so that the failure of an architectural, mechanical, or electrical component shall not cause the failure of a nearby critical architectural, mechanical, or electrical component that is essential to remain in place due to the potential hazard it may cause.

3.1.1 REFERENCES AND STANDARDS: The following references and standards are to be considered part of these provisions to the extent referred to in this chapter:

Ref. 3-1 American Petroleum Institute (API), API STD 650, *Welded Steel Tanks For Oil Storage*, 1988.

Ref. 3-2 American Society of Mechanical Engineers (ASME), ASME A17.1, *Safety Code For Elevators And Escalators*, 1993.

Ref. 3-3 American Society of Mechanical Engineers (ASME), ASME B31, *Code For Pressure Piping*, including addendums through 1993.

Ref. 3-4 American Society of Mechanical Engineers (ASME), *Boiler And Pressure Vessel Code*, including addendums through 1993.

Ref. 3-5 American Society For Testing And Materials (ASTM), ASTM C635, *Standard Specification For The Manufacture, Performance, and Testing of Metal Suspension Systems For Acoustical Tile And Lay-in Panel Ceilings*, 1991.

Ref. 3-6 American Society For Testing And Materials (ASTM), ASTM C636, *Standard Practice For Installation Of Metal Ceiling Suspension Systems For Acoustical Tile And Lay-in Panels*, 1992.

Ref. 3-7 American Water Works Association (AWWA), D100, *Welded Steel Tanks For Water Storage*, 1984.

Ref. 3-8 Ceilings and Interior Systems Construction Association (CISCA), *Recommendations for Direct-Hung Acoustical Tile and Lay-In Panel Ceilings, Seismic Zones 0-2*, 1991.

Ref. 3-9 Ceilings and Interior Systems Construction Association (CISCA), *Recommendations for Direct-Hung Acoustical Tile and Lay-In Panel Ceilings, Seismic Zones 3-4*, 1990.

Ref. 3-10 Institute of Electrical and Electronic Engineers (IEEE), Standard 344, *Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations*, 1987.

Ref. 3-11 Manufacturers Standardization Society of the Valve and Fitting Industry (MSS), SP-58, *Pipe Hangers and Supports--Materials, Design, and Manufacture*, 1988.

Ref. 3-12 National Fire Protection Association (NFPA), NFPA-13, *Standard for the Installation of Sprinkler Systems*. 1991.

Ref. 3-13 Rack Manufacturers Institute (RMI), *Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*, 1990.

Ref. 3-14 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), *HVAC Duct Construction Standards, Metal and Flexible*, 1985.

Ref. 3-15 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), *Rectangular Industrial Duct Construction Standards*, 1980.

Ref. 3-16 Sheet Metal and Air Conditioning Contractors National Association (SMACNA), Sheet Metal Industry Fund of Los Angeles, and Plumbing and Piping Industry Council, *Guidelines for Seismic Restraint of Mechanical Systems and Plumbing Piping Systems*, 1992.

3.1.2 COMPONENT FORCE TRANSFER: Components shall be attached such that the component forces are transferred to the structure of the building. Component seismic attachments shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity.

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

3.1.3 SEISMIC FORCES: Seismic forces (F_p) shall be determined in accordance with Eq. 3.1.3-1:

$$F_p = 4.0 C_a I_p W_p \qquad (3.1.3\text{-}1)$$

Alternatively, F_p may be computed in accordance with Eq. 3.1.3-2 through Eq. 3.1.3-5:

$$F_p = \frac{a_p A_p I_p W_p}{R_p} \qquad (3.1.3\text{-}2)$$

$$A_p = C_a + (A_r - C_a)\left(\frac{x}{h}\right) \qquad (3.1.3\text{-}3)$$

$$A_r = 2.0 A_s \leq 4.0 C_a \qquad (3.1.3\text{-}4)$$

$$F_{p(minimum)} = 0.5 C_a I_p W_p \qquad (3.1.3\text{-}5)$$

where:

F_p = Seismic design force centered at the component's center of gravity and distributed relative to component's mass distribution.

a_p = Component amplification factor that varies from 1.00 to 2.50 (select appropriate value from Table 3.2.2 or Table 3.3.2).

A_p = Component acceleration coefficient (expressed as a fraction of gravity) at point of attachment to structure.

I_p = Component importance factor that is either 1.00 or 1.50 (see Sec. 3.1.5).

W_p = Component operating weight.

R_p = Component response modification factor that varies from 1.50 to 6.00 (select appropriate value from Table 3.2.2 or Table 3.3.2).

C_a = Seismic coefficient at grade as determined from Sec. 1.4.2.3 or Table 1.4.2.4a (expressed as a fraction of gravity).

A_r = Component acceleration coefficient (expressed as a fraction of gravity) at structure roof level.

x = Elevation in structure of component relative to grade elevation.

h = Average roof elevation of structure relative to grade elevation.

A_s = Structure response acceleration coefficient (expressed as a fraction of gravity):

$$A_s = \frac{1.2C_v}{T^{2/3}} \le 2.5C_a \qquad (3.1.3\text{-}7)$$

Note that A_s shall be calculated for each principle horizontal direction of the structure. The <u>largest</u> value for A_s shall be utilized in determining A_r.

C_v = Seismic coefficient as determined from Sec. 1.4.2.3 or Table 1.4.2.4b (expressed as a fraction of gravity).

T = Effective fundamental period of the structure as determined in Sec. 2.4.2.1 and 2.4.3.1.

The force (F_p) shall be applied independently vertically, longitudinally, and laterally in combination with service loads associated with the component.

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

3.1.4 SEISMIC RELATIVE DISPLACEMENTS: Seismic relative displacements (D_p) shall be determined in accordance with the following equations:

For two connection points on the same building or structural system, use the smaller of:

$$D_p = \delta_{xA} - \delta_{yA} \qquad (3.1.4\text{-}1)$$

or

$$D_p = (X - Y)\frac{\Delta_{aA}}{h_{sx}} \quad (3.1.4\text{-}2)$$

For two connection points on separate buildings or structural systems, use the smaller of:

$$D_p = |\delta_{xA}| + |\delta_{yB}| \quad (3.1.4\text{-}3)$$

or

$$D_p = \frac{X\Delta_{aA}}{h_{sx}} + \frac{Y\Delta_{aB}}{h_{sx}} \quad (3.1.4\text{-}4)$$

where:

D_p = Relative seismic displacement that the component must be designed to accommodate.

δ_{xA} = Deflection at building level x of Building A, determined by an elastic analysis as defined in Sec. 2.2.7.1 and multiplied by the C_d factor.

δ_{yA} = Deflection at building level y of Building A, determined by an elastic analysis as defined in Sec. 2.2.7.1 and multiplied by the C_d factor.

δ_{xB} = Deflection at building level x of Building B, determined by an elastic analysis as defined in Sec. 2.2.7.1 and multiplied by the C_d factor.

δ_{yB} = Deflection at building level y of Building B, determined by an elastic analysis as defined in Sec. 2.2.7.1 and multiplied by the C_d factor.

X = Height of upper support attachment at level x as measured from grade.

Y = Height of lower support attachment at level y as measured from grade.

Δ_{Aa} = Allowable story drift for Building A as defined in Table 2.2.7.

Δ_{aB} = Allowable story drift for Building B as defined in Table 2.2.7.

h_{sx} = Story height used in the definition of the allowable drift, Δ_a, in Table 2.2.7. Note that Δ_a/h_{sx} = the allowable drift index.

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

3.1.5 COMPONENT IMPORTANCE FACTOR: The component importance factor (I_p) shall be selected as follows:

$I_p = 1.5$ Life-safety component is required to function after an earthquake.

$I_p = 1.5$ Component contains material that would be significantly hazardous if released.

$I_p = 1.5$ Component poses a significant life-safety hazard if separated from primary structure (e.g., parapets, exterior wall panels).

$I_p = 1.5$ Component can block a means of egress or exitway if damaged (e.g., exit stairs).

$I_p = 1.0$ All other components.

3.2 ARCHITECTURAL COMPONENT DESIGN:

3.2.1 GENERAL: Architectural systems, components, or elements (hereinafter referred to as "components") listed in Table 3.2.2 and their attachments shall meet the requirements of Sec. 3.2.2 through Sec. 3.2.9.

3.2.2 ARCHITECTURAL COMPONENT FORCES AND DISPLACEMENTS: Architectural components shall meet the force requirements of Sec. 3.1.3 and Table 3.2.2.

Components supported by chains or otherwise "hung" from above are not required to meet the seismic force requirements of this section provided that they cannot significantly damage any other component when subject to seismic motion <u>and</u> they have ductile or articulating connections to the structure at the point of attachment.

TABLE 3.2.2
Architectural Component Coefficients

Architectural Component or Element	a_p [a]	R_p [b]
Interior Nonbearing Walls and Partitions		
Stair and elevator enclosures	1	3.0
Other vertical enclosures	1	3.0
Area separation walls or fire walls	1	3.0
Plain (unreinforced) masonry walls	1	1.5
All other walls and partitions	1	3.0
Cantilever Elements		
Parapets	2.5	1.5
Chimneys	2.5	1.5
Stacks	2.5	3.0
Exterior Nonbearing Walls	1[d]	3.0

TABLE 3.2.2 continued

Architectural Component or Element	a_p [a]	R_p [b]
Exterior Wall Panels		
Panel	1	3.0
Connecting members and fasteners	1[d]	3.0
Veneer		
Ductile materials and attachments	1	4.0
Nonductile materials and attachments	1	1.5
Penthouses	2.5	4.0
Ceilings		
All	1	1.5
Racks and Cabinets		
Storage racks more than 8 ft (2.4 m) in height	2.5	4.0 [c]
Storage racks detailed in accordance with the provisions of Chapter 5	2.5	6.0 [c]
Storage cabinets and laboratory equipment	1	3.0
Access Floors		
Special access floors (designed in accordance with Sec. 3.2.7.2)	2.5	6.0
All other	2.5	3.0
Appendages and Ornamentations	1	3.0
Other Rigid Components		
Ductile materials and attachments	1	4.0
Nonductile materials and attachments	1	1.5
Other Flexible Components		
Ductile materials and attachments	2.5	4.0
Nonductile materials and attachments	2.5	1.5

[a] A lower value for a_p may be justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See the "Glossary" for definitions of rigid and flexible components including attachments.

[b] $R_p = 1.5$ for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow (nonductile) cast-in-place anchors or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in Seismic Performance Categories D or E. Shallow anchors are those with an embedment length-to-bolt diameter ratio of less than 8.

[c] Storage racks over 8 ft in height shall be designed in accordance with the provisions of Sec. 3.2.9.1.

[d] Where flexible diaphragms provide lateral support for walls and partitions, the value of a_p shall be increased to 2.0 for the center one-half of the span.

3.2.3 ARCHITECTURAL COMPONENT DEFORMATION: Architectural components that could pose a life-safety hazard shall be designed for the seismic relative displacement requirements of Sec. 3.1.4. Architectural components shall be designed for vertical deflection due to joint rotation of cantilever structural members.

3.2.4 EXTERIOR WALL PANEL CONNECTIONS: Exterior nonbearing wall panels that are attached to or enclose the structure shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4. In addition, movements of the structure resulting from temperature changes shall be accommodated. Such elements shall be supported by means of structural connections or by mechanical connections and fasteners. The support system shall be designed in accordance with the following:

a. Connections and panel joints shall allow for the story drift caused by relative seismic displacements (D_p) determined in Sec. 3.1.4.

b. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections that provide equivalent sliding or ductile capacity.

c. The connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

d. The body of the connecting member shall have a design strength to resist the force (F_p) determined in Sec. 3.1.3 assuming a_p equals 1.0 and R_p equals 3.0.

e. All fasteners in the connecting system such as bolts, inserts, welds, and dowels shall be designed for the force (F_p) determined in Sec. 3.1.3 assuming a_p equals 1.0 and R_p equals 1.0.

3.2.5 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 Sec. 3.1.3 shall not exceed the deflection capability of the component or system.

3.2.6 SUSPENDED CEILINGS: Suspended ceilings shall meet all the requirements of Sec. 3.2.6.1, 3.2.6.2, and 3.2.6.7 and any of the methods specified in Sec. 3.2.6.3, 3.2.6.4, 3.2.6.5, or 3.2.6.6.

3.2.6.1 Seismic Forces: Unless exempted by Sec. 3.2.6.4.1 or compliance with Sec. 3.2.6.5 is demonstrated, suspended ceilings shall be designed to meet the force provisions of Sec. 3.1.3 and the additional provisions of the following sections.

The weight of the ceiling, W_p, shall include the ceiling grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid; and other components which are laterally supported by the ceiling.

The seismic force, F_p, shall be transmitted through the ceiling attachments to the building structural elements or the ceiling-structure boundary.

Design of anchorage and connections shall be in accordance with these provisions.

For ceiling areas exceeding 2,500 ft^2 (232 m^2), analyses shall be performed to determine if seismic separation joints in the ceiling system are required.

3.2.6.2 Installation: The manufacture and installation of suspended ceilings shall meet the requirements of Ref. 3-5 and Ref. 3-6 unless the design approach specified in Sec. 3.2.6.6 is used. Conduit, piping, ducts, and cabling shall be supported independently of the suspended ceiling system, including vertical supports and bracing elements. Flexible connections shall be used for all ductwork interfaces with the ceiling grid. Diagonal bracing is permitted for mechanical or electrical components provided the design of the bracing is considered with the lateral force resisting system of the ceiling grid.

3.2.6.3 Industry Standard Construction: Suspended ceilings may be designed and constructed in accordance with the following sections.

3.2.6.3.1: Suspended ceilings in Seismic Performance Categories B and C shall be designed and installed in accordance with the Ceilings and Interior Systems Construction Association (CISCA) recommendations for seismic zones 0-2 (Ref. 3-8), except that seismic forces shall be determined in accordance with Sec. 3.1.3 and 3.2.6.1.

3.2.6.3.2: Suspended ceilings in Seismic Performance Categories D and E shall be designed and installed in accordance with the Ceilings and Interior Systems Construction Association (CISCA) recommendations for seismic zones 3-4 (Ref. 3-9), except that seismic forces shall be determined in accordance with Sec. 3.1.3 and 3.2.6.1. In addition, if lateral bracing wires are provided, compression struts between the runners and the supporting structure shall be provided at the points of wire attachment to the runners.

3.2.6.3.3 Sprinkler heads and other penetrations in Seismic Performance Categories B and C shall have a minimum of 1/4 in. (6 mm) clearance on all sides. Sprinkler heads and other penetrations in Seismic Performance Categories D and E shall have a minimum of 1/2 in. (13 mm) clearance on all sides.

Smaller gaps around penetrations may be used if it can be demonstrated that the penetration stem has adequate flexibility to accommodate 1/2 in. (12 mm) of displacement without exceeding allowable stresses.

3.2.6.4 Unbraced Construction: Suspended ceilings may be designed and constructed in accordance with the following sections. Ceiling panels shall be designed such that the edges of the panels are positioned on the closure angle such that they have a maximum wall clearance of $1/4w$ all around and an object clearance, c, around all ceiling penetrations where w and c are as defined in the following paragraphs.

3.2.6.4.1: Ceiling systems which meet both of the following requirements need not be designed for seismic forces or displacements:

$$w \geq \frac{C_v\sqrt{gh}}{10} \qquad (3.2.6.4.1\text{-}1)$$

$$c \geq \frac{C_v\sqrt{gh}}{40} \qquad (3.2.6.4.1\text{-}2)$$

where:

w = perimeter width in length units, the width of the supporting closure angle at the perimeter of the ceiling system, usually at a wall surface, building column, or seismic separation joints. Ceiling panels shall be designed such that the edges of the panels are positioned on the closure angle such that they have a wall clearance of $3/8w$ all around.

c = object clearance in length units, the clear dimension between the ceiling panels and stiff penetrating objects such as sprinkler heads or HVAC grills.

h = plenum height in length units, the length of the supporting member from the bottom of the ceiling system to the supporting structure attachment point.

g = acceleration of gravity in length units per second per second.

3.2.6.4.2: Ceiling systems which do not meet both of the requirements of Sec. 3.2.6.4.1 shall be designed such that both of the following requirements are met:

$$w \geq 2\Delta \qquad (3.2.6.4.2\text{-}1)$$

$$c \geq 0.5\Delta \qquad (3.2.6.4.2\text{-}2)$$

where w and c are as defined above and Δ = calculated suspended ceiling lateral deflection, in length units, with respect to the supporting structure when the seismic forces of Sec. 3.2.6.1 are applied at the suspended ceiling elevation.

3.2.6.5 Braced Construction: Suspended ceilings may be designed and constructed in accordance with the following sections.

3.2.6.5.1: Where substantiating seismic force calculations are not provided, horizontal restraints shall be effected by four 0.12 in (3 mm) wires secured to the main runner within 2 in. (51 mm) of the cross-runner intersection and splayed 90 degrees from each other at an angle not exceeding 45 degrees from the plane of the ceiling. These horizontal restraint points shall be placed approximately 12 ft (3.6 m) on center in both directions with the first placement no more than 6 ft (1.8 m) from each wall. These horizontal restraint points shall be in a non-symmetrical pattern to minimize diaphragm loads.

The perimeter width (w) of the supporting closure angle shall be a minimum of 2 in. (51 mm) in Seismic Performance Categories D and E and a minimum of 1 in. (25 mm) in Seismic Performance Categories B and C. In Seismic Performance Categories D and E, compression struts shall be located at each horizontal restraint point.

Sprinkler heads and other penetrations in Seismic Performance Categories B and C shall have a minimum of 1/2 in. (13 mm) clearance on all sides. Sprinkler heads and other penetrations in Seismic Performance Categories D and E shall have a minimum of 1 in. (25 mm) clearance on all sides.

Smaller gaps around penetrations may be used if it can be demonstrated that the penetration stem has adequate flexibility to accommodate 1/2 in. (13 mm) of displacement without exceeding allowable stresses.

3.2.6.5.2: Rigid braces may be used instead of the described wires. Attachment of the bracing elements to the structure above shall be adequate to limit lateral deflections to less than 1/4 in. (6 mm) for the loads prescribed in Sec. 3.1.3. Sprinkler heads and other penetrations shall have a minimum of 1/4 in. (6 mm) clearance on all sides.

3.2.6.5.3: Lateral force bracing members shall be spaced a minimum of 6 in. (150 mm) from all piping or duct work that is not provided with its own bracing restraints for horizontal forces. Bracing members shall be attached to the grid and to the supporting structure in such a manner that they can provide support for the horizontal seismic design force or 200 lb (900 N), whichever is greater. Lateral force bracing for ceilings with plenum depths exceeding 8 ft (2.5 m), measured from the supporting structure to the grid surface, shall be designed and detailed to meet the force and displacement requirements of Sec. 3.1.3 and 3.1.4.

3.2.6.6 Integral Ceiling/Sprinkler Construction: As an alternate to providing large clearances around sprinkler system penetrations through ceiling systems, the sprinkler system and ceiling grid may be designed and tied together as an integral unit. Such a design shall consider the mass and flexibility of all elements involved, including: ceiling system, sprinkler system, light fixtures, and mechanical (HVAC) appurtenances. The design shall be performed by a registered engineer.

3.2.6.7 Partitions: In Seismic Performance Categories D and E, ceiling bracing members shall not be used to brace ceiling-high partitions and partitions penetrating the ceiling.

3.2.7 ACCESS FLOORS:

3.2.7.1 General: Access floors shall be designed to meet the force provisions of Sec. 3.1.3 and the additional provisions of this section. The weight of the access floor, W_p, shall include the weight of the floor system, 100 percent of the weight of all equipment fastened to the floor, and 25 percent of the weight of all equipment supported by, but not fastened to the floor. The seismic force, F_p, shall be transmitted from the top surface of the access floor to the supporting structure.

Overturning effects of equipment <u>fastened</u> to the access floor panels also shall be considered. The ability of "slip on" heads for pedestals shall be evaluated for suitability to transfer overturning effects of equipment.

When checking individual pedestals for overturning effects, the <u>maximum</u> concurrent axial load shall not exceed the portion of W_p assigned to the pedestal under consideration.

3.2.7.2 Special Access Floors: Access floors shall be considered to be "special access floors" if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, concrete anchors, welding, or bearing. Design load capacities comply with recognized design codes and/or certified test results.

2. Seismic loads are not transmitted by friction produced solely by the effects of gravity, powder-actuated fasteners (shot pins), or adhesives.

3. The bracing system shall be designed considering the destabilizing effects of individual members buckling in compression.

4. Bracing and pedestals are of structural or mechanical shape produced to ASTM specifications that specify minimum mechanical properties. Electrical tubing shall not be used.

5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

3.2.8 PARTITIONS: Partitions that are tied to the ceiling and all partitions greater than 6 ft (1.8 m) in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling splay bracing. Bracing shall be spaced to limit horizontal deflection at the partition head to be compatible with ceiling deflection requirements as determined in Sec. 3.2.6 for suspended ceilings and Sec. 3.2.2 for other systems.

3.2.9 STEEL STORAGE RACKS:

3.2.9.1 At Grade Elevation: Storage racks installed at grade elevation shall be designed, fabricated, and installed in accordance with Ref. 3-13 and the following requirements:

a. If designed as a building structure, requirements of Chapter 2 shall be met. R_p shall be taken as 4. Higher values of R_p may be used if supported by test results or if the full detailing requirements of Chapters 2 and 5 are met.

b. If designed as a architectural component or system, seismic design forces shall not be less than that required by Sec. 3.1.3.

c. Dead load weight (W) in seismic force calculations shall not be less than the weight of the storage rack plus 67 percent of the rated load of the rack placed on all levels.

3.2.9.2 Above-Grade Elevations: Storage racks installed at elevations above-grade shall be designed, fabricated, and installed in accordance with Ref. 3-13 and the following requirements:

a. Storage racks shall meet the force and displacement requirements of Sec. 3.1.3 and 3.1.4.

b. Dead load weight (W) in seismic force calculations shall not be less than the weight of the storage rack plus 67 percent of the rated load of the rack placed on all levels.

3.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN:

3.3.1 GENERAL: Attachments and equipment supports for the mechanical and electrical systems, components, or elements (hereinafter referred to as "components") shall meet the requirements of Sec. 3.3.2 through Sec. 3.3.16.

3.3.2 MECHANICAL AND ELECTRICAL COMPONENT FORCES AND DISPLACEMENTS: Mechanical and electrical components shall meet the force and seismic relative displacement requirements of Sec. 3.1.3, Sec. 3.1.4, and Table 3.3.2.

Some complex equipment such as valves and valve operators, turbines and generators, and pumps and motors may be functionally connected by mechanical links not capable of transferring the seismic loads or accommodating seismic relative displacements and may require special design considerations such as a common rigid support or skid.

Components supported by chains or otherwise "hung" from above are not required to meet the seismic force requirements of this section provided that they cannot significantly damage any other component when subject to seismic motion and they have ductile or articulating connections to the building at the point of attachment.

TABLE 3.3.2
Mechanical and Electrical Components Coefficients

Mechanical and Electrical Component or Element	a_p [a]	R_p [b]
General Mechanical Equipment		
Boilers and furnaces	1.0	3.0
Pressure vessels on skirts and free-standing	2.5	3.0
Stacks	2.5	3.0
Cantilevered chimneys	2.5	1.5
Other	1.0	3.0
Manufacturing and Process Machinery		
General	1.0	3.0
Conveyors (nonpersonnel)	2.5	3.0
Piping Systems (see also Sec. 3.3.11)	2.5	4.0
Storage Tanks and Spheres		
Flat bottom (anchored)	2.5	4.0
Flat bottom (unanchored)	2.5	3.0
On braced or unbraced legs	2.5	2.0

TABLE 3.3.2 continued

Mechanical and Electrical Component or Element	a_p [a]	R_p [b]
HVAC System Equipment		
Vibration isolated	2.5	3.0
Nonvibration isolated	1.0	3.0
Mounted in-line with ductwork	1.0	3.0
Other	1.0	3.0
Elevator Components	1.0	3.0
Trussed Towers (free-standing or guyed)	2.5	3.0
General Electrical Equipment		
Communication	1.0	3.0
Bus ducts, conduit, cable tray	2.5	6.0
Panelboards, battery racks	2.5	3.0
Motor control centers, switchgear	2.5	3.0
Other	1.0	3.0
Lighting Fixtures	1.0	1.5

[a] A lower value for a_p may be justified by detailed dynamic analyses. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for equipment generally regarded as flexible or flexibly attached. See the "Glossary" for definitions of rigid and flexible components including attachments.

[b] $R_p = 1.5$ for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow nonductile cast-in-place anchors or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in Seismic Performance Categories D or E. Shallow anchors are those with an embedment length-to-bolt diameter ratio of less than 8.

3.3.3 MECHANICAL AND ELECTRICAL COMPONENT PERIOD: The fundamental period of the mechanical and electrical component (and its attachment to the building), T_p, may be determined by the following equation provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single-degree-of-freedom system:

$$T_p = 2\pi \sqrt{\frac{W_p}{K_p g}} \tag{3.3.3}$$

where:

T_p = Component fundamental period,

W_p = Component operating weight,

g = Gravitational acceleration, and

K_p = Stiffness of resilient support system of the component and attachment, determined in terms of load per unit deflection at the center of gravity of the component.

Note that consistent units must be used.

Alternatively, determine the fundamental period of the component in seconds (T_p) from experimental test data or by a properly substantiated analysis.

3.3.4 MECHANICAL AND ELECTRICAL COMPONENT ATTACHMENTS: The stiffness of mechanical and electrical component attachments shall be designed such that the load path for the component performs its intended function.

3.3.5 COMPONENT SUPPORTS: Mechanical and electrical component supports and the means by which they are attached to the component shall be designed for the forces determined in Sec. 3.1.3 and in conformance with Chapters 5 through 9, as appropriate, for the materials comprising the means of attachment. Such supports include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers. Component supports may be forged or cast as a part of the mechanical or electrical component. If standard or proprietary supports are used, they shall be designed by either load rating (i.e., testing) or for the calculated seismic forces. In addition, the stiffness of the support, when appropriate, shall be designed such that the seismic load path for the component performs its intended function.

Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Sec. 3.1.4.

In addition, the means by which supports are attached to the component, except when integral (i.e., cast or forged), shall be designed to accommodate both the forces and displacements determined in accordance with Sec. 3.1.3 and 3.1.4. If the value of $I_p = 1.5$ for the component, the local region of the support attachment point to the component shall be evaluated for the effect of the load transfer on the component wall.

3.3.6 COMPONENT CERTIFICATION: The manufacturer's certificate of compliance with the force requirements of the *Provisions* shall be submitted to the regulatory agency when required by the contract documents or when required by the regulatory agency.

3.3.7 UTILITY AND SERVICE LINES AT BUILDING INTERFACES: At the interface of adjacent structures or portions of the same building that may move independently, utility lines shall be provided with adequate flexibility to accommodate the anticipated differential movement.

3.3.8 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. 1.4.3.1. Specific attention shall be given to the vulnerability of underground utilities and utility interfaces between the structure and the ground in all situations where Soil Profile Type E or F is present and where the seismic coefficient C_a is equal to or greater than 0.15.

3.3.9 STORAGE TANKS:

3.3.9.1 Above-Grade Storage Tanks: Attachments and supports for storage tanks mounted above grade in buildings or structures shall be designed to meet the force provisions of Sec. 3.1.3. The weight of the storage tank (W_p) shall include the weight of the tank structure and appurtenances and the operating weight of the contents at maximum rated capacity.

3.3.9.2 At-Grade Storage Tanks: Flat-bottom storage tanks mounted at grade shall be designed to meet the force provisions of either Ref. 3-1, Ref. 3-7, or Sec. 3.1.3.

In addition, tanks designated with an I_p of 1.5 or tanks greater than 20 ft (6.2 m) in diameter or tanks that have a height-to-diameter ratio greater than 1.0 shall also be designed to meet the following addition requirements:

1. Sloshing effects shall be considered.

2. Piping connections to steel storage tanks shall consider the potential uplift of the tank wall during earthquakes. Unless otherwise calculated, the following displacements shall be assumed for all side-wall connections and bottom penetrations:

    a. Vertical displacement of 2 in. (51 mm) for anchored tanks,

    b. Vertical displacement of 12 in. (305 mm) for unanchored tanks, and

    c. Horizontal displacement of 8 in. (203 mm) for unanchored tanks with a diameter of 40 ft (12.2 m) or less.

3.3.10 HVAC DUCTWORK: Attachments and supports for HVAC ductwork systems shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section. In addition to their attachments and supports, ductwork systems designated as having an $I_p = 1.5$ themselves shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section.

Seismic restraints are not required for HVAC ducts with $I_p = 1.0$ if either of the following conditions are met:

a. HVAC ducts are suspended from hangers 12 in. (305 mm) or less in length from the top of the duct to the supporting structure. The hangers shall be detailed to avoid significant bending of the hangers.

or

b. HVAC ducts have a cross-sectional area of less than 6 ft^2 (0.557 m^2).

HVAC duct systems fabricated and installed in accordance with the SMACNA duct construction standards (Ref. 3-14 and Ref. 3-15) shall be deemed to meet the lateral bracing requirements of this section.

Equipment items installed in-line with the duct system (e.g., fans, heat exchangers, and humidifiers) weighing more than 75 lb (334 N) shall be supported and laterally braced independent of the duct system and shall meet the force requirements of Sec. 3.1.3.

3.3.11 PIPING SYSTEMS: Attachments and supports for piping systems shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section. In addition to their attachments and supports, piping systems designated as having I_p = 1.5 themselves shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section.

Seismic effects that shall be considered in the design of a piping system include the dynamic effects of the piping system, its contents, and, when appropriate, its supports. The interaction between the piping system and the supporting structures, including other mechanical and electrical equipment shall also be considered.

3.3.11.1 Pressure Piping Systems: Pressure piping systems designed and constructed in accordance with Ref. 3-3 shall be deemed to meet the force, displacement, and other provisions of this section. In lieu of specific force and displacement provisions provided in the ASME B31, the force and displacement provisions of Sec. 3.1.3 and 3.1.4 shall be used. Special provisions for hydraulic elevator pipe are provided in Chapters 3 and 24 of Ref. 3-2.

3.3.11.2 Fire Protection Sprinkler Systems: Fire protection sprinkler systems designed and constructed in accordance with Ref. 3-12 shall be deemed to meet the force, displacement, and other requirements of this section.

3.3.11.3 Other Piping Systems: Piping designated as having an I_p = 1.5 but not designed and constructed in accordance with Ref. 3-3 or Ref. 3-12 shall meet the following:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

   (1) For piping systems constructed with ductile materials (e.g., steel, aluminum or copper), 90 percent of the piping material yield strength.

   (2) For threaded connections with ductile materials, 70 percent of the piping material yield strength.

   (3) For piping constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25 percent of the piping material minimum specified tensile strength.

   (4) For threaded connections in piping constructed with nonductile materials, 20 percent of the piping material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for piping components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Piping shall be investigated to ensure that the piping has adequate flexibility between support attachment points to the structure, ground, other mechanical and electrical equipment, or other piping.

d. Piping shall be investigated to ensure that the interaction effects between it and other piping or constructions are acceptable.

3.3.11.4 Supports and Attachments for Other Piping: Attachments and supports for piping not designed and constructed in accordance with Ref. 3-3 or Ref. 3-12 shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural code such as, when constructed of steel, Ref. 5-1 and 5-2 or Ref. 3-11.

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Rod hangers may be considered seismic supports if the length of the hanger from the supporting structure is 12 in. (305 mm) or less. Rod hangers shall not be constructed in a manner that would subject the rod to bending moments.

d. Seismic supports are not required for:

(1) Ductile piping in Seismic Performance Category D or E designated as having an $I_p = 1.5$ and a nominal pipe size of 1 in. (25 mm) or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.

(2) Ductile piping in Seismic Performance Category B or C designated as having an $I_p = 1.5$ and a nominal pipe size of 2 in. (51 mm) or less when provisions are made to protect the piping from impact or to avoid the impact of larger piping or other mechanical equipment.

(3) Ductile piping in Seismic Performance Category D or E designated as having an $I_p = 1.0$ and a nominal pipe size of 3 in. (76 mm) or less.

(4) Ductile piping in Seismic Performance Category A, B, or C designated as having an $I_p = 1.0$ and a nominal pipe size of 6 in. (152 mm) or less.

e. Seismic supports shall be constructed so that support engagement is maintained.

3.3.12 BOILERS AND PRESSURE VESSELS: Attachments and supports for boilers and pressure vessels shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section. In addition to their attachments and supports, boilers and pressure vessels designated as having an $I_p = 1.5$ themselves shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4.

Seismic effects that shall be considered in the design of a boiler or pressure vessel include the dynamic effects of the boiler or pressure vessel, its contents, and its supports; sloshing of liquid contents; loads from attached components such as piping; and the interaction between the boiler or pressure vessel and its support.

3.3.12.1 ASME Boilers and Pressure Vessels: Boilers or pressure vessels designed in accordance with Ref. 3-4 shall be deemed to meet the force, displacement, and other requirements of this section. In lieu of the specific force and displacement provisions provided in the ASME code, the force and displacement provisions of Sec. 3.1.3 and 3.1.4 shall be used.

3.3.12.2 Other Boilers and Pressure Vessels: Boilers and pressure vessels designated as having an $I_p = 1.5$ but not constructed in accordance with the provisions of Ref. 3-4 shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

   (1) For boilers and pressure vessels constructed with ductile materials (e.g., steel, aluminum or copper), 90 percent of the material minimum specified yield strength.

   (2) For threaded connections in boilers or pressure vessels or their supports constructed with ductile materials, 70 percent of the material minimum specified yield strength.

   (3) For boilers and pressure vessels constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25 percent of the material minimum specified tensile strength.

   (4) For threaded connections in boilers or pressure vessels or their supports constructed with nonductile materials, 20 percent of the material minimum specified tensile strength.

b. Provisions shall be made to mitigate seismic impact for boiler and pressure vessel components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Boilers and pressure vessels shall be investigated to ensure that the interaction effects between them and other constructions are acceptable.

3.3.12.3 Supports and Attachments for Boilers and Pressure Vessels: Attachments and supports for boilers and pressure vessels shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with nationally recognized structural code such as, when constructed of steel, Ref. 5-1 and 5-2.

b. Attachments embedded in concrete shall be suitable for cyclic loads.

c. Seismic supports shall be constructed so that support engagement is maintained.

3.3.13 MECHANICAL EQUIPMENT ATTACHMENTS AND SUPPORTS: Attachments and supports for mechanical equipment not covered in Sec. 3.3.8 through 3.3.12 or 3.3.16 shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section. In addition to their attachments and supports, such mechanical equipment designated as having an $I_p = 1.5$, itself, shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section.

Seismic effects that shall be considered in the design of mechanical equipment, attachments and their supports include the dynamic effects of the equipment, its contents, and when appropriate its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered.

3.3.13.1 Mechanical Equipment: Mechanical equipment designated as having an $I_p = 1.5$ shall meet the following provisions.

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects such as corrosion shall not exceed the following:

   (1) For mechanical equipment constructed with ductile materials (e.g., steel, aluminum, or copper), 90 percent of the equipment material minimum specified yield strength.

   (2) For threaded connections in equipment constructed with ductile materials, 70 percent of the material minimum specified yield strength.

   (3) For mechanical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25 percent of the equipment material minimum tensile strength.

   (4) For threaded connections in equipment constructed with nonductile materials, 20 percent of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. The possibility for loadings imposed on the equipment by attached utility or service lines due to differential motions of points of support from separate structures shall be evaluated.

3.3.13.2 Attachments and Supports for Mechanical Equipment: Attachments and supports for mechanical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural standard specification such as, when constructed of steel, Ref. 5-1 and 5-2.

b. Friction clips shall not be used for anchorage attachment.

c. Expansion anchors shall not be used for mechanical equipment rated over 10 hp (7.45 Kw).

EXCEPTION: Undercut expansion anchors may be used.

d. Drilled and grouted-in-place anchors for tensile load applications shall use either expansive cement or expansive epoxy grout.

e. Supports shall be specifically evaluated if weak-axis bending of cold-formed support steel is relied on for the seismic load path.

f. Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$.

g. . Seismic supports shall be constructed so that support engagement is maintained.

3.3.14 ELECTRICAL EQUIPMENT ATTACHMENTS AND SUPPORTS: Attachments and supports for electrical equipment shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section. In addition to their attachments and supports, electrical equipment designated as having I_p = 1.5, itself, shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 and the additional provisions of this section.

Seismic effects that shall be considered in the design of other electrical equipment include the dynamic effects of the equipment, its contents, and when appropriate its supports. The interaction between the equipment and the supporting structures, including other mechanical and electrical equipment, shall also be considered.

3.3.14.1 Electrical Equipment: Electrical equipment designated as having an I_p = 1.5 shall meet the following provisions:

a. The design strength for seismic loads in combination with other service loads and appropriate environmental effects shall not exceed the following:

   (1) For electrical equipment constructed with ductile material (e.g. steel, aluminum, or copper), 90 percent of the equipment material minimum specified yield strength.

   (2) For threaded connections in equipment constructed with ductile materials, 70 percent of the material minimum specified yield strength.

   (3) For electrical equipment constructed with nonductile materials (e.g., plastic, cast iron, or ceramics), 25 percent of the equipment material minimum tensile strength.

   (4) For threaded connections in equipment constructed with nonductile materials, 20 percent of the material minimum specified yield strength.

b. Provisions shall be made to mitigate seismic impact for equipment components constructed of nonductile materials or in cases where material ductility is reduced (e.g., low temperature applications).

c. Evaluate loading on the equipment imposed by attached utility or service lines which are also attached to separate structures.

d. Batteries on racks shall have wrap-around restraints to ensure that the batteries will not fall off the rack. Racks shall be evaluated for sufficient lateral and longitudinal load capacity.

e. Internal coils of dry type transformers shall be positively attached to their supporting substructure within the transformer enclosure.

f. Slide out components in electrical control panels shall have a latching mechanism to hold contents in place.

g. Electrical cabinet design shall conform to the applicable National Electrical Manufacturers Association (NEMA) standards. Cut-outs in the lower shear panel that are larger than 6 in. (152 mm) wide and 12 in. (305 mm) high shall be specifically evaluated if an evaluation is not provided by the manufacturer.

h. The attachment of additional items weighing more than 100 lb (445 N) shall be specifically evaluated if not provided by the manufacturer.

3.3.14.2 Attachments and Supports for Electrical Equipment: Attachments and supports for electrical equipment shall meet the following provisions:

a. Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and designed and constructed in accordance with a nationally recognized structural standard specification such as, when constructed of steel, Ref. 5-1 and 5-2.

b. Friction clips shall not be used for anchorage attachment.

c. Oversized washers shall be used at bolted connections through the base sheet metal if the base is not reinforced with stiffeners.

d. Supports shall be specifically evaluated if weak-axis bending of light gage support steel is relied on for the seismic load path.

e. The supports for linear electrical equipment such as cable trays, conduit, and bus ducts shall be designed to meet the force and displacement provisions of Sec. 3.1.3 and 3.1.4 only if any of the following situations apply:

Supports are cantilevered up from the floor;

Supports include bracing to limit deflection;

Supports are constructed as rigid welded frames;

Attachments into concrete utilize nonexpanding insets, shot pins, or cast iron embedments; or

Attachments utilize spot welds, plug welds, or minimum size welds as defined by AISC.

3.3.15 ALTERNATIVE SEISMIC QUALIFICATION METHODS: As an alternative to the analysis methods implicit in the design methodology described above, equipment testing is an acceptable method to determine seismic capacity. Thus, adaptation of a nationally recognized standard, such as Ref. 3-10, is acceptable so long as the equipment seismic capacity equals or exceeds the demand expressed in Sec. 3.1.3 and 3.1.4.

3.3.16 ELEVATOR DESIGN REQUIREMENTS: Elevators shall meet the force and displacement provisions of Sec. 3.3.2 unless exempted by either Sec. 1.2 or Sec. 3.1. Elevators designed in accordance with the seismic provisions of Ref. 3-2 shall be deemed to meet the seismic force requirements of this section, except they also shall meet the additional requirements provided in Sec. 3.3.16.1 through 3.3.16.4..

3.3.16.1 Elevators and Hoistway Structural System: Elevators and hoistway structural systems shall be designed to meet the force and displacement provisions of Sec. 3.3.2.

3.3.16.2 Elevator Machinery and Controller Supports and Attachments: Elevator machinery and controller supports and attachments shall be designed to meet the force and displacement provisions of Sec. 3.3.2.

3.3.16.3 Seismic Controls: Seismic switches shall be provided for all elevators addressed by Sec. 3.3.16.1 including those meeting the requirements of the ASME reference, provided they operate with a speed of 150 ft/min (46 m/min) or greater. Seismic switches shall provide an electrical signal indicating that structural motions are of such a magnitude that the operation of elevators may be impaired. Upon activation of the seismic switch, elevator operations shall conform to provisions in Ref. 3-2 except as noted below. The seismic switch shall be located at or above the highest floor serviced by the elevators. The seismic switch shall have two horizontal perpendicular axes of sensitivity. Its trigger level shall be set to 30 percent of the acceleration of gravity

In facilities where the loss of the use of an elevator is a life-safety issue, the elevator may be used after the seismic switch has triggered provided that:

1. The elevator shall operate no faster than the service speed,

2. The elevator shall be operated remotely from top to bottom and back to top to verify that it is operable, and

3. The individual putting the elevator back in service should ride the elevator from top to bottom and back to top to verify acceptable performance.

3.3.16.4 Retainer Plates: Retainer plates are required at the top and bottom of the car and counterweight.

Chapter 4

FOUNDATION DESIGN REQUIREMENTS

4.1 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.

4.2 STRENGTH OF COMPONENTS AND FOUNDATIONS: The resisting capacities of the foundations, subjected to the prescribed seismic forces of Chapters 1 and 2, shall meet the requirements of this chapter.

4.2.1 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 5, 6, 7, 8, or 9. The strength of foundation components shall not be less than that required for forces acting without seismic forces.

4.2.2 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. 2.2.6, 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.

4.3 SEISMIC PERFORMANCE CATEGORIES A AND B: Any construction meeting the requirements of Sec. 4.1 and 4.2 may be used for buildings assigned to Category A or B.

4.4 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.

4.4.1 INVESTIGATION: The regulatory agency may require the submission of a written report that shall include, in addition to the requirements of Sec. 4.1 and the evaluations required in Sec. 4.2.2, 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.

4.4.2 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.

4.4.3 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 product of the larger pile cap or column load times the seismic coefficient C_a divided by 4 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.

4.4.4 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. 6-1. 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.

4.4.4.1 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 ft (3 m) below the ground. There shall be a minimum of four bars with closed ties (or equivalent spirals) of a minimum 1/4 in. (6 mm) diameter provided at 16-longitudinal-bar-diameter maximum spacing with a maximum spacing of 4 in. (102 mm) in the top 2 ft (0.6 m) of the pile. Reinforcement detailing requirements shall be in conformance with Sec. 6.6.2.

4.4.4.2 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.

4.4.4.3 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.

4.4.4.4 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 in. (102 mm) in the top 2 ft (0.6 m). Reinforcement shall be full length.

4.4.4.5 Precast-Prestressed Piles: The upper 2 ft (0.6 m) of the pile shall have No. 3 ties minimum at not over 4-in. (102 mm) spacing or equivalent spirals. The pile cap connection may be by means of dowels as required in Sec. 4.4.4. Pile cap connection may be by means of developing pile reinforcing strand if a ductile connection is provided.

4.5 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.

4.5.1 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.

4.5.2 FOUNDATION TIES: Individual spread footings founded on soil defined in Sec. 1.4.2 as Soil Profile Type E or F shall be interconnected by ties. Ties shall conform to Sec. 4.4.3.

4.5.3 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 Type E or F. Piles subject to such deformation shall be designed and detailed in accordance with Sec. 5.6.3 or 6.3.3 for a length equal to 120 percent of the flexural length (point of fixity to pile cap).

4.5.3.1 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 ft (3 m) 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 in. (76 mm) in the top 4 ft (1.2 m) of the pile. Ties shall be a minimum of No. 3 bars for up to 20-in.-diameter (500 mm) piles and No. 4 bars for piles of larger diameter.

4.5.3.2 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.

4.5.3.3 Precast Concrete Piles: Ties in precast concrete piles shall conform to the requirements of Chapter 6 for at least the top half of the pile.

4.5.3.4 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. 6-1, 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.

4.5.3.5 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 5

STEEL STRUCTURE DESIGN REQUIREMENTS

5.1 REFERENCE DOCUMENTS: The design, construction, and quality 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. 5-1 *Load and Resistance Factor Design Specification for Structural Steel Buildings (LRFD)*, American Institute of Steel Construction (AISC), December 1993

Ref. 5-2 *Allowable Stress Design and Plastic Design Specification for Structural Steel Buildings (ASD)*, American Institute of Steel Construction, June 1, 1989

Ref. 5-3 *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, June 1992

Ref. 5-4 *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. 5-5 *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members*, American Iron and Steel Institute (AISI), March 16, 1991

Ref. 5-6 *Specification for the Design of Cold-formed Stainless Steel Structural Members*, ANSI/ASCE 8-90, American Society of Civil Engineers

Ref. 5-7 *Standard Specification, Load Tables and Weight Tables for Steel Joists and Joist Girders*, Steel Joist Institute, 1992 Edition

Ref. 5-8 *The Criteria for Structural Applications for Steel Cables for Buildings*, AISI, 1973 Edition.

5.2 STEEL STRUCTURE DESIGN REQUIREMENTS: The design of structural steel members and connections to resist seismic forces shall be in accordance with Ref. 5-1 or 5-2. When using the provisions of Ref. 5-2 to compute the capacity of members to resist seismic forces, allowable stresses shall be converted into design strengths using the provisions of Part II, Sec. 3.2 and 3.3, of Ref. 5-3. When required, structural steel members also shall be designed in accordance with Ref. 5-3 as modified by the requirements of this section except that the definition of E shall be as defined in these provisions and that the term C_a shall be substituted for A_v throughout.

Also, Sec. 8.2.c of Ref. 5-3 shall be deleted and replaced with the following:

"**8.2.c Connection Strength:** Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic testing results or calculation, the ability to sustain inelastic rotation and to develop the strength criteria in Sec. 8.2.a considering the expected value of yield strength and strain hardening." (See *Commentary* Sec. 5.2.)

Also, Sec. 8.2.d of Ref. 5.3 shall be deleted.

5.2.1 REQUIREMENTS FOR SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF): SCBFs shall be designed in accordance with the requirements of Ref. 5-3 for concentrically braced frames except as modified herein. The reference to section and paragraph numbers are those of Ref. 5-3. The following modifications shall apply to SCBFs and shall not modify the requirements for ordinary concentrically braced frames in Ref. 5-3:

1. Sec. 9.2.a--Shall not apply to SCBF.

2. Sec. 9.2.b--Revise as follows:

   "**9.2.b Compressive Design Strength:** The design strength of a bracing member in axial compression shall not exceed $\phi_c P_n$."

3. Sec. 9.2.d--Revise as follows:

   "**9.2.d Width-thickness Ratio:** Width-thickness ratios of stiffened and unstiffened compression elements of braces shall comply with Sec. B5 of Ref. 5-1. Braces shall be compact (i.e., $\lambda < \lambda_p$). The width-thickness ratio of angle sections shall not exceed $52/\sqrt{F_y}$ (the metric equivalent is $137/\sqrt{F_y}$ where F_y is in MPa). Circular sections shall have an outside diameter to wall thickness ratio not exceeding $1{,}300/F_y$ (the metric equivalent is $8960/F_y$ where F_y is in MPa); rectangular tubes shall have outside wall width-thickness ratio not exceeding $110/\sqrt{F_y}$ (the metric equivalent is $289/\sqrt{F_y}$ where F_y is in MPa) unless the curved section or tube walls are stiffened."

4. Sec. 9.2.e--Revise as follows:

   "**9.2.e Built-up Member Stitches:** For all built up braces, the spacing of stitches shall be uniform and not less than two stitches shall be used:

   "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 tensile strength of each element.

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

5. Sec. 9.4.a--Revise as follows:

"**9.4.a V and Inverted V Type Bracing:** V braced and inverted V braced frames shall comply with the following:

"1. A beam intersected by braces shall be continuous between columns.

"2. A beam intersected by braces shall be capable of supporting all tributary dead and live loads assuming the bracing is not present.

"3. A beam intersected by braces shall be capable of resisting the combination of load effects by Eq. 3-5 and 3-6 of Ref 5.3 except that the term Q_b shall be substituted for the term E where Q_b = the maximum unbalanced load effect applied to the beam by the braces. This load effect shall be permitted to be calculated using a minimum of P_y for the brace in tension and a maximum of $0.3\phi_c P_n$ for the brace in compression.

"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, $F_y b_f t_f$."

6. Sec. 9.4.b--Delete in its entirety without replacement.

7. Sec. 9.5--Delete in its entirety without replacement.

8. Add a new section as follows:

"**9.5 Columns:**

"**9.5.a Compactness:** Columns used in SCBFs shall be compact according to Sec. B5 of the *Specification*. The outside wall width-thickness ratio of rectangular tubes used for columns shall not exceed $110/\sqrt{F_y}$ (the metric equivalent is $9.13/\sqrt{F_y}$ where F_y is in MPa) unless otherwise stiffened.

"**9.5.b Splices:** In addition to meeting the requirements of Sec. 6.2, column splices in SCBFs also shall be designed to develop the nominal shear strength and 50 percent of the nominal moment strength of the section."

5.3 COLD-FORMED STEEL SEISMIC REQUIREMENTS: The design of cold-formed carbon or low-alloy steel to resist seismic loads shall be in accordance with the provisions of Ref.

5-4, 5-5, and 5-6 except as modified by this section. The reference to section and paragraph numbers are to those of the particular specification modified.

5.3.1 REF. 5-4: The nominal strength of members and connections shall be as specified therein except that the nominal strength for shear and web crippling shall be determined by multiplying the allowable strength by 1.7. Design strengths shall be determined by multiplying the nominal strengths by resistance factors as stated herein. The following resistance factors, ϕ, shall be used:

Shear strength with $h/t > (Ek_v/F_y)^{1/2}$	$\phi = 0.9$
Shear strength with $h/t \leq (Ek_v/F_y)^{1/2}$	$\phi = 1.0$
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	$\phi = 1.55/\Omega$

5.3.2 REF. 5-4: Revise Sec. A4.4 as follows:

"**A4.4 Wind or Earthquake Loads** Where load combinations specified by the applicable code include wind loads, the resulting forces may be multiplied by 0.75. Seismic load combinations shall be as determined by these provisions."

5.3.3 REF. 5-5: Modify Sec. A5.1.4 by substituting a load factor of 1.0 in place of 1.5 for nominal earthquake load.

5.3.4 REF. 5-6: Modify Sec. 1.5.2 by substituting a load factor of 1.0 in place of 1.5 for nominal earthquake load.

5.4 SEISMIC REQUIREMENTS FOR STEEL DECK DIAPHRAGMS: Steel deck diaphragms shall be made from materials conforming to the requirements of Ref. 5-4, 5-5, or 5-6. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a licensed design professional experienced in testing of cold-formed steel assemblies and approved by the regulatory agency having jurisdiction. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, ϕ, equal to 0.60 (for mechanically connected diaphragms) and equal to 0.50 (for welded diaphragms). The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.

5.5 STEEL CABLES: The design strength of steel cables shall be determined by the provisions of Ref. 5-8 except as modified by these provisions. Sec. 5d of Ref. 5-8 shall be modified by substituting $1.5(T_4)$ where 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. 5-8.

5.6 SEISMIC PROVISIONS FOR STEEL STRUCTURAL MEMBERS: Steel structures and structural elements therein that resist seismic forces shall be designed in accordance with the applicable provisions of Sec. 5.2, 5.3, 5.4, and 5.5. In addition, steel structures also shall be designed in accordance with the requirements of Sec. 5.6.1, 5.6.2, and 5.6.3 for the appropriate Seismic Performance Category.

5.6.1 SEISMIC PERFORMANCE CATEGORIES A AND B: Buildings assigned to Seismic Performance Categories A and B shall be of any construction permitted by the references in Sec. 5.1

5.6.2 SEISMIC PERFORMANCE CATEGORY C: Unless otherwise required by the provisions of this section, Seismic Performance Category C buildings shall be of any construction permitted by the references in Sec. 5.1.

5.6.3 SEISMIC PERFORMANCE CATEGORY D AND E: Buildings assigned to Seismic performance categories D and E shall be designed in accordance with the additional provisions of Ref. 5-3 for structural steel buildings and Sec. 5.7 for light-framed walls.

5.7 LIGHT-FRAMED WALLS: Cold-formed steel stud walls designed in accordance with Ref. 5-4, 5-5, and 5-6 shall, when required by the provisions of Sec. 5.6.3, also comply with the requirements of this section.

5.7.1 BOUNDARY MEMBERS: All boundary members, chords, and collectors shall be designed to transmit the axial forces induced by the specified loads of these provisions.

5.7.2 CONNECTIONS: Connections for diagonal bracing members, top chord splices, boundary members, and collectors shall have a design strength equal to or greater than the nominal tensile strength of the members being connected or $2R/5$ times the design seismic force. The term $2R/5$ shall not be taken less than unity. The pull-out resistance of screws shall not be used to resist seismic forces.

5.7.3 BRACED BAY MEMBERS: In systems where the lateral forces are resisted by braced frames, the vertical and diagonal members in braced bays shall be anchored such that the tracks are not required to resist tensile forces by bending of the track or track web. Both flanges of studs shall be braced to prevent lateral torsional buckling.

5.7.4 DIAGONAL BRACES: Provision shall be made for pretensioning or other methods of installation of tension-only bracing to prevent loose diagonals.

Chapter 6

CONCRETE STRUCTURE DESIGN REQUIREMENTS

6.1 REFERENCE DOCUMENTS: The quality and testing of concrete and steel materials and the design and construction of concrete components that resist seismic forces shall conform to the requirements of the references listed in this section except as modified by the provisions of this chapter.

Ref. 6-1 Building Code Requirements for Reinforced Concrete, American Concrete Institute, ACI 318-89 (Revised 1992), excluding Appendix A

Ref. 6-2 Building Code Requirements for Structural Plain Concrete, ACI 318.1-89 (Revised 1992)

6.1.1 MODIFICATIONS TO REF. 6-1:

6.1.1.1: Replace Sec. 9.2.3 with Sec. 2.2.6 of this document.

6.1.1.2: Insert the following definitions in Sec. 21.1:

"**Connection** - Interface where two precast concrete elements or a precast concrete element and a cast-in-place element are interconnected.

"**Coupling Beam** - A horizontal element in plane with and connecting two adjacent structural walls.

"**Joint** - For monolithic reinforced concrete structures and precast concrete structural systems satisfying 21.2.2.5 the joint is the geometric volume common to the intersecting members. Effective area of joint for frame for shear is defined in 21.5.3.

"**Joint Region - Frames** - The joint plus the volumes within lengths equal to twice the intersecting members' depths measured from the faces of the joint.

"**Joint Region - Panels** - The joint plus the volumes within lengths equal to twice the intersecting members' thickness measured from the faces of the joint.

"**Non-Linear Action Location** - Center of plastic hinge zone, line of shear slip or midpoint of extending element."

6.1.1.3: Replace Sec. 21.2.1.3 and 21.2.1.4 with the provisions of this chapter.

6.1.1.4: Insert the following new Sec. 21.2.1.6:

"A reinforced concrete structural system composed of precast elements shall be permitted for the seismic force resisting system if:

"1. It emulates the behavior of monolithic reinforced concrete construction and satisfies 21.2.2.5, or

"2. It relies on the unique properties of a structural system composed of interconnected precast elements and it is demonstrated by experimental evidence or analysis to safely sustain the seismic loading requirements of a comparable monolithic reinforced concrete structure satisfying Chapter 21."

6.1.1.5: Insert the following new Sec. 21.2.2.5, 21.2.2.6 and 21.2.2.7:

"21.2.2.5 Structural systems emulating the behavior of reinforced concrete construction and composed of precast concrete elements shall satisfy 21.2.2.6 or 21.2.2.7.

"21.2.2.6 Precast concrete structural systems shall utilize strong connections resulting in nonlinear response remote from those connections. Designs shall satisfy the requirements of 21.2.7 in addition to the requirements of 21.2 through 21.6, as applicable. For 21.4.1 the restrictions on cross-sectional dimensions and clear span to effective depth shall also apply to the dimensions and span between nonlinear response locations.

"21.2.2.7 If 21.2.2.6 is not satisfied systems shall utilize connections that result in joint regions having characteristics providing performance for the structure equal to or exceeding that for a comparable monolithic reinforced concrete structure. In addition, designs shall satisfy the requirements of 21.2.8 as well as the requirements of 21.2 through 21.6, as applicable."

6.1.1.6: Insert the following new Sec. 21.2.5.2 and 21.2.5.3:

"21.2.5.2 Prestressing 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 the greater of 350 psi or $f'_c/12$ at locations of nonlinear action.

"21.2.5.3 For members in which prestressing tendons are used together with ASTM A706 or A615 (Grades 40 or 60) reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more that 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 tensile strength of the tendon. Tendons shall extend through exterior joints and be anchored at the exterior face or beyond."

6.1.1.7: Insert the following new Sec. 21.2.7:

"21.2.7 Members resisting earthquake induced forces in precast concrete frames using strong connections shall satisfy the following conditions:

"21.2.7.1 Locations for non-linear action shall be selected so that there is a strong column/weak beam deformation mechanism under earthquake effects. For beam to continuous column connections, the locations shall not be closer to the column than three quarters of the depth of the beam. For beam to beam connections, the location shall be permitted to be anywhere within the flexural members of the frames but shall be no closer to the connection than three quarters of the depth of the beam. For column to continuous beam connections and column to column connections, locations shall be permitted to be anywhere within the beam length outside the joint. For column to footing connections where energy dissipation is required at the column base to complete the non-linear deformation mechanism, the locations for non-linear action shall be no closer to the footing than three quarters of the width of the column in the direction of the mechanism considered.

"21.2.7.2 Design strength of cross sections of connections shall be based on

$$\phi S_n^{CONNECTION} \geq \frac{11}{10} S_n^{FRAME} \tag{21-A}$$

where $S_n^{CONNECTION}$ is nominal strength (moment or shear) at connection cross-section, ϕ is strength reduction factor for that force, and S_n^{FRAME} is the force, (moment or shear), determined from consideration of the probable resistances at the locations for non-linear action required by 21.2.7.1.

"21.2.7.3 In addition to 21.2.7.2 at column to column connections the design moment strength ϕM_n shall not be less than 0.4 times the maximum M_{pr} for the column within the story height."

6.1.1.8: Insert the following new Sec. 21.2.8

"21.2.8 Precast concrete frame and wall systems designed using 21.2.2.7 shall satisfy the following conditions:

"21.2.8.1 The deformed shape of the structure under specified lateral loads shall emulate that for the same structure constructed in monolithic reinforced concrete.

"21.2.8.2 The main longitudinal reinforcement of frame members, and the boundary reinforcement for walls, shall be made continuous across connections and able to develop a stress of at least $1.25f_y$ in tension and compression.

"21.2.8.3 Design of connection cross sections shall be based on the assumption that the capacity for connection moment, M_{pr}, is its probable strength in flexure or the capacity for connection shear, S_{pr}, is its probable strength in shear.

"21.2.8.4 When connection capacity is M_{pr} the co-existing applied shear, S, must be not greater than $\phi\, S_n^{CONNECTION}$ times a non-linear action modification factor, 0.5 δ_s where δ_s is a factor for reversed cyclic loading, varying linearly between 1.0 for a shear that reverses to not greater than 50 percent of its maximum value and 4/5 for a shear that fully reverses.

"21.2.8.5 When connection capacity is S_{pr} the co-existing applied moment, M, must be not greater than $\phi M_n^{CONNECTION}$.

"21.2.8.6 Where the connection is subject to an axial force in addition to shear and moment that axial force shall be considered in computing the probable and nominal strengths in 21.2.8.4 and 21.2.8.5.

"21.2.8.7 The probable moment strength M_{pr} of the connection shall be determined using a strength reduction factor of 1.0 and a reinforcing steel stress of at least 1.25 f_y.

"21.2.8.8 The probable shear strength S_{pr} for shear slip shall be taken as the strength calculated from Sec. 11.7 times δ_s.

"21.2.8.9 The probable and nominal shear strengths for the connection shall be less than the corresponding shear strengths immediately adjacent to the connection of the members meeting at the connection."

6.1.1.9: 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 (75 mm) extensions shall be spaced not more than $d/2$ throughout the length of the member."

6.1.1.10: Add the following new Sec. 21.4.5.3:

"At any section where the design strength, ϕP_n, of the column is less than the sum of the shear V_e computed in accordance with Sec. 21.4.5.1 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."

6.1.1.11: Change the reference to Sec. 9.2 in Sec. 21.6.3 to the load combination specified in Sec. 2.2.6 of this document for earthquake forces.

6.1.1.12: Add a new Sec. 21.6.6 as follows and renumber existing Sec. 21.6.6 through 21.6.8 to 21.6.7 through 21.6.9:

"**21.6.6.1** A cast-in-place reinforced concrete slab used as a diaphragm to resist earthquake forces shall not be less than 2-inches (50 mm) thick.

"**21.6.6.2** A cast-in-place reinforced topping slab bonded to a precast floor or roof system and used as a diaphragm to resist earthquake forces shall comply with all of the following:

1. The topping slab shall not be less than 2-1/2 inches (63 mm) thick;
2. Its connections shall be proportioned and detailed to provide a complete load path for transfer of shear to chords, collectors, and resisting elements.
3. Bonding of the topping slab to the precast concrete elements shall be in accordance with Sec. 11.7.9.

6.1.1.13: Add a new Sec. 21.6.7 as follows and renumber Sec. 21.6.6 through 21.6.7 as modified by Sec. 6.1.1.12 to 21.6.8 through 21.6.10.

"**21.6.7 Coupling Beams:**

"**21.6.7.1:** For coupling beams with $l_n/d \geq 4$, the design shall conform to the requirements of Sec. 21.2 and 21.3 for structures in Seismic Performance Category D or E or to Sec. 21.8 for structures in Seismic Performance Category C. It shall be permitted to waive the requirements of Sec. 21.3.1.3 and 21.3.1.4 if it can be shown by rational analysis that lateral stability is adequate or if alternative means of maintaining lateral stability is provided.

"**21.6.7.2:** Coupling beams with $l_n/d < 4$ shall be permitted to be reinforced with two intersecting groups of symmetrical diagonal bars. Coupling beams with $l_n/d < 4$ and with factored shear force V_u exceeding $4\sqrt{f_c'}\, b_w d$ (the metric equivalent is $0.332\sqrt{f_c'}\, b_w d$ where f_c' is in MPa and b_w and D are in mm)shall be reinforced with two intersecting groups of symmetrical diagonal bars. Each group shall consist of a minimum of four bars assembled in a core each side of which is a minimum of $b_w/2$. The design shear strength, ϕV_n, of these coupling beams shall be determined by:

$$\phi V_n = 2\phi f_y \sin\alpha \, A_{vd} \leq 10\phi\sqrt{f_c'}\, b_w d \quad (21.5)$$

where:

α = the angle between the diagonal reinforcement and the longitudinal axis,

A_{vd} = total area of reinforcement in each group of diagonal bars, and

ϕ = 0.85.

The metric equivalent of Eq. 21.5 is as follows:

$$0.83\,\phi\sqrt{f_c'}\;b_w d$$

where f_c' is in mm.

> **EXCEPTION:** The design of coupling beams need not comply with the requirements for diagonal reinforcement if it can be shown that failure of the coupling beams will not impair the vertical load carrying capacity of the structure, the egress from the structure, or the integrity of nonstructural components and connections or produce other unacceptable effects. The analysis shall take into account the changes of stiffness of the structure due to the failure of coupling beams. Design strength of coupling beams assumed to be part of the seismic force resisting system shall not be reduced below the values otherwise required.

"**21.6.7.3:** 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 as per Eq. 10-5 and 21-3, the minimum cover as specified in Sec. 7.7 shall be assumed over each group of diagonally placed reinforcing bars.

"**21.6.7.4:** Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to Sec. 10.5, 11.8.9, and 11.8.10.

"**21.6.7.5:** Contribution of the diagonal reinforcement to nominal flexural strength of the coupling beam area shall be considered.

6.1.1.14: Change the title of Sec. 21.8 to read: "Requirements for Intermediate Moment Frames."

6.1.2 MODIFICATIONS TO REF. 6-2:

6.1.2.1: Amend Sec. 1.2.3 to read:

> "Plain concrete shall not be used for structural members where special design considerations are required for blast unless explicitly permitted by the legally adopted general building code."

6.2 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. 6.2.2.

6.2.1 LOAD FACTOR MULTIPLIERS: In addition to the load factors in Sec. 2.2.6, 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. 2.2.6 shall have a multiplier of 3 if special inspection is not provided or of 2 if it is provided.

6.2.2 STRENGTH OF ANCHORS: Strength of anchors cast in concrete shall be taken as the lesser of the strengths associated with concrete failure and the anchor steel failure. Where feasible, anchor connections, particularly those subject to seismic or other dynamic loads, shall be designed and detailed such that the connection failure is initiated by failure of the anchor steel rather than by failure of the surrounding concrete. Reinforcement also shall be permitted to be used for direct transfer of tension and shear loads. Such reinforcement shall be designed with proper consideration of its development and its orientation with respect to the postulated concrete failure planes.

The strength of headed bolts and headed studs cast in concrete shall be based on testing in accordance with Sec. 6.2.3 or calculated in accordance with Sec. 6.2.4. The bearing area of headed anchors shall be at least one and one-half times the shank area.

6.2.3 STRENGTH BASED ON TESTS: The strength of anchors shall be based on not less than 10 representative tests conforming to the proposed materials and anchor size and type, embedment length, and configuration as to attachment plates, loads applied, and concrete edge distances. The nominal strength shall be the mean value derived from the tests minus one standard deviation. The strength reduction factor applied to the nominal strength shall be 0.8 when anchor failure governs in the majority of tests and 0.65 when concrete failure controls.

6.2.4 STRENGTH BASED ON CALCULATIONS: Calculations for design strength shall be in accordance with Sec. 6.2.4.1 through 6.2.4.3.

6.2.4.1 Strength in Tension: The design tensile strength of the individual anchors or adequately connected groups of anchors shall be the minimum of P_s or ϕP_c where:

1. Design tensile strength governed by steel, P_s, in pounds (N), is:

$$P_s = 0.9 A_b F_u n \tag{6.2.4.1-1}$$

where:

A_b = the area, in.^{2} (mm^2) of the shank of the bolt or stud,

n = the number of anchors, and

F_u = the specified ultimate tensile strength, psi (MPa), of the anchor. A307 bolts or A108 studs are permitted to be assumed to have F_u of 60,000 psi (414 Mpa).

2. Design tensile strength governed by concrete failure, ϕP_c in pounds (N) is as follows:

   a. For individual anchors or groups of anchors with individual anchors spaced at least twice their embedment length apart and spaced not less than one anchor embedment length from a free edge of the concrete:

$$\phi P_c = \phi \lambda \sqrt{f_c'}(2.8 A_s) n \tag{6.2.4.1-2}$$

where:

A_s = area (in.^{2}) of the assumed failure surface taken as a truncated cone sloping at 45 degrees from the head of the anchor to the concrete surface as shown in Figure 6.2.4.1a;

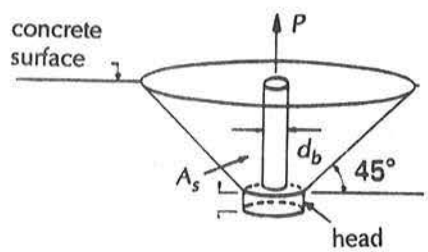

FIGURE 6.2.4.1a Shear cone failure for a single headed anchor.

f_c' = concrete strength (psi)--6,000 psi (41 MPa) maximum;

ϕ = strength reduction factor of 0.65 except that where special transverse reinforcing is provided to confine the concrete engaged by the anchor and is extended to pass through the failure surface into adjacent concrete, ϕ is permitted to be taken as 0.85;

λ = lightweight concrete factor--1 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for lightweight concrete.

The metric equivalent of Eq. 6.2.4.1-2 is:

$$\phi P_c = \frac{\phi \lambda \sqrt{f_c'}(2.8 A_s) n}{12}$$

where A_s is in mm^2 and f_c' is MPa.

Where any anchors are closer to a free edge of the concrete than the anchor embedment length, the design tensile strength of those anchors shall be reduced proportionately to the edge distance divided by the embedment length. For multiple edge distances less than the embedment length, use multiple reductions.

b. For anchor groups where individual anchors are spaced closer together than two embedment lengths:

$$\phi P_c = \phi \lambda \sqrt{f_c'}(2.8 A_p + 4 A_t) \qquad (6.2.4.1\text{-}3)$$

where:

A_p = area (in.^{2}) of an assumed failure surface taken as a truncated pyramid extending from the heads of the outside anchors in the group at 45 degrees to the concrete surface as shown in Figure 6.2.4.1b;

A_t = area (in.^{2}) of the flat bottom surface of the truncated pyramid of the assumed concrete failure surface shown in Figure 6.2.4.1b.

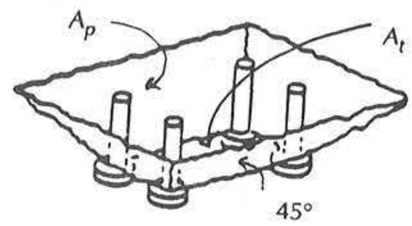

FIGURE 6.2.4.1b Truncated pyramid failure for a group of headed anchors.

The metric equivalent of Eq. 6.2.4.1-3 is:

$$\phi P_c = \frac{\phi \lambda \sqrt{f'_c}(2.8A_p + 4A_t)}{12}$$

where A_p and A_t are in mm^2 and f'_c is in MPa.

If any anchors are closer to a free edge of the concrete than the anchor embedment length, the design tensile strength shall be reduced by using the reduced area A_p in the equation above.

Anchor groups shall be checked for a critical failure surface passing completely through a concrete member along the 45 degree lines as shown in Figure 6.2.4.1c with $A_t = 0$ and A_p based on the area of the sloping failure surface passing completely through the concrete member. The lowest allowable load shall govern.

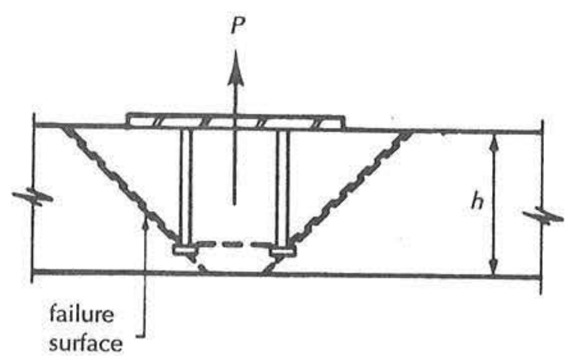

FIGURE 6.2.4.1c Pull-out failure surface for a group of headed anchors in thin section.

6.2.4.2 Strength in Shear: The design shear strength of anchors shall be the minimum of V_s or ϕV_c where the design shear strength governed by steel failure is V_s, in pounds (N), and the design shear strength governed by concrete failure is ϕV_c, in pounds (N). In situations where the embedment and/or concrete edge distances are limited, reinforcement to confine concrete to preclude its premature failure shall be permitted.

a. Where anchors are loaded toward an edge with edge distance d_e from the back row of anchors as shown in Figure 6.2.4.2 equal to or greater than 15 anchor diameters and the distance from the front row of anchors to the edge equal to or greater than 6 anchor diameters:

$$V_s = (0.75 A_b F_u) n \tag{6.2.4.2-1}$$

$$\phi V_c = (\phi 800 A_b \lambda \sqrt{f_c'}) n \tag{6.2.4.2-2}$$

where:

A_b = the area, in.^{2} (mm^2) of the shank of the bolt or stud;

F_u = the specified ultimate tensile strength (psi) of the anchor. A307 bolts or A108 studs are permitted to be assumed to have F_u of 60,000 psi (414 MPa);

n = the number of anchors;

λ = lightweight concrete factor--1 for normal weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for lightweight concrete; and

f_c' = concrete strength (psi)--6,000 psi (41 MPa) maximum.

The metric equivalent of Eq. 6.2.4.2-2 is:

$$\phi V_c = \frac{\left(\phi 800 A_b \lambda \sqrt{f_c'}\right) n}{12}$$

where A_b is in mm^2 and f_c' is in MPa.

b. Where anchors are loaded toward an edge with d_e less than 15 anchor diameters or the front row closer to the edge than 6 anchor diameters:

$$V_s = (0.75 A_b F_u) n_b \tag{6.2.4.2-3}$$

$$\phi V_c = \phi V_c' C_w C_t C_c \qquad (6.2.4.2\text{-}4)$$

where:

A_b = the area (in.^{2}) of the shank of the bolt or stud.

F_u = the specified ultimate tensile strength (psi) of the anchor. A307 bolts or A108 studs are permitted to be assumed to have F_u of 60,000 psi (414 MPa).

n_b = the number of anchors in the back row.

$\phi V'_c$= the design shear strength of an anchor in the back row:

$$\phi V_c' = \phi\, 12.5 d_e^{1.5} \lambda \sqrt{f_c'} \le 800 A_b \lambda \sqrt{f_c'} \qquad (6.2.4.2\text{-}5)$$

where d_e = the distance from the anchor axis to the free edge (in.).

C_w = the adjustment factor for group width:

$$C_w = 1 + \left(\frac{b}{3.5 d_e}\right) \le n_b \qquad (6.2.4.2\text{-}6)$$

where b = the center-to-center distance of outermost anchors in the back row (see Figure 6.2.4.2) (in.) and d_e = the distance from the anchor axis to the free edge (in.).

C_t = the adjustment factor for member thickness:

$$C_t = \frac{h}{1.3 d_e} \le 1.0 \qquad (6.2.4.2\text{-}7)$$

where h = the thickness of concrete (in.) and d_e is as above.

C_c = the adjustment factor for member corner effects:

$$C_c = 0.4 + 0.7\left(\frac{d_c}{d_e}\right) \le 1.0 \qquad (6.2.4.2\text{-}8)$$

where d_c = the distance, measured perpendicular to the load, from the free edge of the concrete to the nearest anchor in in. (see Figure 6.2.4.2) and d_e is as above.

The metric equivalent of Eq. 6.2.4.2-5 is:

$$\phi V_c' = \frac{\phi\, 12.5 d_e^{1.5} \lambda \sqrt{f_c'}}{2.39} \leq \frac{\phi\, 800 A_b \lambda \sqrt{f_c'}}{12}$$

where d_e is in mm, A_b is in mm^2 and f_c' is in MPa.

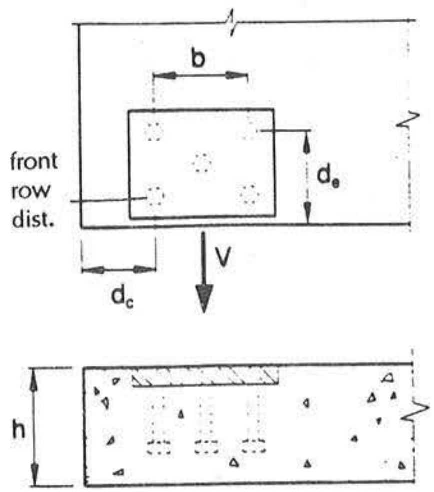

FIGURE 6.2.4.2 Shear on a group of headed anchors.

6.2.4.3 Combined Tension and Shear: Where tension and shear act simultaneously, all of the following conditions must be met:

$$\frac{1}{\phi}\left(\frac{V_u}{V_c}\right) \leq 1.0 \qquad (6.2.4.3\text{-1a})$$

$$\frac{1}{\phi}\left(\frac{P_u}{P_c}\right) \leq 1.0 \qquad (6.2.4.3\text{-1b})$$

$$\frac{1}{\phi}\left[\left(\frac{P_u}{P_c}\right)^2 + \left(\frac{V_u}{V_c}\right)^2\right] \leq 1.0 \qquad (6.2.4.3\text{-2a})$$

$$\left(\frac{P_u}{P_s}\right)^2 + \left(\frac{V_u}{V_s}\right)^2 \leq 1.0 \qquad (6.2.4.3\text{-}2b)$$

where:

P_u = required tensile strength, in pounds (N), based on factored loads and

V_u = required shear strength, in pounds (N), based on factored loads.

6.3 CLASSIFICATION OF MOMENT FRAMES:

6.3.1 ORDINARY MOMENT FRAMES: Ordinary moment frames are frames conforming to the requirements of Ref. 6-1 exclusive of Chapter 21.

6.3.2 INTERMEDIATE MOMENT FRAMES: Intermediate moment frames are frames conforming to the requirements of Sec. 21.8 of Ref. 6-1 in addition to those requirements for ordinary moment frames.

6.3.3 SPECIAL MOMENT FRAMES: Special moment frames are frames conforming to the requirements of Sec. 21.2 through 21.5 of Ref. 6-1 in addition to those requirements for ordinary moment frames.

6.4 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A may be of any construction permitted in Ref. 6-1 and 6-2 and these provisions.

6.5 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.

6.5.1 ORDINARY MOMENT FRAMES: In flexural members of ordinary moment frames forming part of the seismic-force-resisting system, at least two main flexural reinforcing bars shall be provided continuously top and bottom throughout the beams, through or developed within exterior columns or boundary elements.

Columns of ordinary moment frames having a clear height to maximum plan dimension ratio of five or less shall be designed for shear in accordance with Sec. 21.8.3 of Ref. 6-1.

6.5.2 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system of a building assigned to Category B and founded on Soil Profile Type E or F shall be intermediate moment frames conforming to Sec. 6.3.2 or special moment frames conforming to Sec. 6.3.3.

6.6 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.

6.6.1 MOMENT FRAMES: All moment frames that are part of the seismic force resisting system shall be intermediate moment frames conforming to Sec. 6.3.2 or special moment frames conforming to Sec. 6.3.3.

6.6.2 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. 21.8.5.1 of Ref. 6-1 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. 6-1.

6.6.3 PLAIN CONCRETE: Structural members of plain concrete in buildings assigned to Category C shall conform to all requirements for Category B and the additional provisions and limitations of this section.

6.6.3.1 Walls: Basement, foundation, or other walls below the base shall be reinforced as required by Sec. 7.1.6.5 of Ref. 6-2. Other walls shall be reinforced as required by Sec. 8.3.7.2.

6.6.3.2 Footings: Isolated footings of plain concrete supporting pedestals or columns are permitted provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

> **EXCEPTION:** In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member shall be permitted to exceed the footing thickness.

Plain concrete footings supporting walls shall be provided with not less than two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. Continuity of reinforcement shall be provided at corners and intersections.

> **EXCEPTION:** In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete footings supporting walls shall be permitted without longitudinal reinforcement.

6.6.3.3 Pedestals: Plain concrete pedestals shall not be used to resist lateral seismic forces.

6.7 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.

6.7.1 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. 6.3.3.

6.7.2 SEISMIC FORCE RESISTING SYSTEM: All materials and components in the seismic force resisting system shall conform to Sec. 21.2 through 21.6 of Ref. 6-1.

6.7.3 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. 2.2.2.4.3 of these provisions and to Sec. 21.7.1.1 or 21.7.1.2 and 21.7.2 of Ref. 6-1.

6.7.4 PLAIN CONCRETE: Structural members of plain concrete are not permitted in buildings assigned to Category D or E.

EXCEPTIONS:

1. In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete footings without longitudinal reinforcement supporting walls and isolated plain concrete footings supporting columns or pedestals are permitted.

2. In all other buildings, plain concrete footings supporting walls are permitted provided they are reinforced longitudinally as specified in Sec. 6.6.3.2.

3. In detached one- and two-family dwellings three stories or less in height and constructed with stud bearing walls, plain concrete foundation or basement walls are permitted provided the wall is not less than 7-1/2 in. (190 mm) thick and retains no more than 4 ft (1219 mm) of unbalanced fill.

Appendix to Chapter 6

REINFORCED CONCRETE STRUCTURAL SYSTEMS COMPOSED FROM INTERCONNECTED PRECAST ELEMENTS

PREFACE: The provisions for reinforced concrete structural systems composed of precast elements in the body of the 1994 *Provisions* are for precast systems emulating monolithic reinforced concrete construction. However, one of the principal characteristics of precast systems is that they often are assembled using dry joints where connections are made by bolting, welding, post-tensioning, or other similar means. Research conducted to date documents concepts for design using dry joints and the behavior of subassemblages composed from interconnected precast elements both at and beyond peak strength levels for nonlinear reversed cyclic loadings (Applied Technology Council, 1981; Cheok and Lew, 1992; Clough, 1986; Eliott et al., 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al., 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al., 1986; Stanton et al., 1991).* This appendix is included for information and as a compilation of the current understanding of the performance under seismic loads of structural systems composed from interconnected precast elements. It is considered premature to base code provisions on this resource appendix; however, user review, trial designs, and comment on this appendix are encouraged. Please direct such feedback to the BSSC.

6A.1 GENERAL:

6A.1.1 SCOPE: Design and construction of lateral force resisting structural systems composed using interconnected precast concrete elements shall comply with the requirements of this appendix. The quality and testing of concrete and steel materials and the design and construction of the precast concrete components and systems that resist seismic forces shall conform to the requirements of the reference document listed in this section except as modified by the provisions of Chapter 6 and this appendix.

6A.1.2 REFERENCE DOCUMENT:

Ref. 6A-1 Building Code Requirements for Reinforced Concrete, American Concrete Institute, ACI 318-89 (Revised 1992), excluding Appendix A.

* See the *Commentary* for this appendix for full reference information.

6A.2 GENERAL PRINCIPLES: A reinforced concrete structural system composed from interconnected precast concrete elements shall be permitted for the lateral force resisting system:

1. If the force-deformation relationships for the connection regions have been validated through physical experiments or the use of analytical models based on the results of physical experiments that closely simulate the building's connection regions and

2. If the response of the building is analyzed using the force-deformation relationships for the connection and joint regions in combination with the force-deformation relationships for the precast concrete elements connected by those regions.

6A.3 LATERAL FORCE RESISTING STRUCTURAL FRAMING SYSTEMS:

6A.3.1: The basic structural and seismic force resisting systems and seismic performance category and building height limitations shall be those specified in Table 2.2.2. The response modification coefficients, R, and the deflection amplification factors, C_d, of Table 2.2.2 shall be taken as maximum values for interconnected construction.

6A.3.2: The response modification coefficients, R, and the deflection amplification factors, C_d, for interconnected construction shall be consistent with the detailing practice for the connections.

6A.3.3: Where force-deformation relationships for the connections have been determined from analytical modeling and have not been validated through physical experiments, R and C_d factors for interconnected construction shall be restricted as shown in Table 6A.3.3.

TABLE 6A.3.3
Restrictions on R and C_d

Restricted Response Modification Coefficient, R_j	Restricted Deflection Amplification Factor, C_{dj}	Seismic Performance Category [a]				Connection Performance Category [b]
		A&B	C	D	E	
$R_j \le R/2$	$C_{dj} \le C_d/2$	P	P	NP	NP	B
$R/2 \le R_j \le R - 1$	$C_d/2 \le C_{dj} \le C_d - 1$	P	P	P	P	C

NOTE: R = R value for monolithic concrete construction in Table 2.2.2 and $C_d = C_d$ value for monolithic concrete construction in Table 2.2.2. R_j and C_{dj} shall be varied in step with R and C_d between limits shown. P = permitted and NP = not permitted.

[a] See Table 1.4.4.

[b] See Sec. 6A.4.3.

6A.3.4: Designs shall provide:

1. A continuous load path to the foundation for all components for seismic forces;

2. Force-deformation relationships for the connection and joint regions that result in a lateral deflection profile for the structure that has deflections increasing continuously with increasing height above the structure's base when a horizontal force is applied in any direction at the top of the structure; and

3. Integrity of the entire load path at deformations C_d times the elastic deformation.

6A.4 CONNECTION PERFORMANCE REQUIREMENTS:

6A.4.1: Connections that are part of the lateral force resisting system and are intended to be nonlinear action locations shall have hinging, sliding, or extending characteristics provided by at least one of the following means:

1. Member hinging in flexure due to reinforcement yield in tension and/or compression or in-plane dry joint opening rotation constrained by yielding in tension or compression of reinforcement crossing that joint.

2. Dry joint movement caused by yield in tension, flexure, or shear of steel plates, bars, or shapes crossing that joint.

3. In-plane dry joint slips caused by shears acting on constrained deformation devices such as friction bolted steel assemblies.

4. Other actions for which physical experiments have established the deformation response of the connection and the region surrounding the connection or joint.

6A.4.2: The seismic performance of a given connection depends on the characteristics of all three of the following:

1. Connector -- The device that crosses the interface between the interconnected precast elements or the cast-in-place element.

2. Anchorage -- The means by which the force in the connector is transferred into the precast or cast-in-place element, and

3. Connection Region -- The volume of element over which the force from the anchorage flows out to match the uniform stress state for the element.

6A.4.3: Based on the results of physical experiments or analytical modeling, nonlinear action, connections and their surrounding connection or joint regions shall be classified into Connection Seismic Performance Categories A, B, and C as follows:

1. For Connection Performance Category A, there shall be no special requirements.

2. For Connection Performance Category B, connections and their surrounding regions shall exhibit stable inelastic reversed cyclic deformation characteristics for the demands placed on them at the R and C_d values selected for the building's seismic force resisting system.

3. For Connection Performance Category C, connections and their surrounding regions shall have stiffness, strength, energy absorption, and energy dissipation capacities that ensure a performance for the building equivalent to that required for the R and C_d values selected for the building's seismic force resisting system.

6A.4.4: For lateral force resisting systems of Seismic Performance Category B, the nonlinear action connections shall be of Connection Performance Category B or C and the anchorage for any such connector transferring tensile or shear force shall be connected directly by welding or similar means or by adequate lap length to the principal reinforcement of the precast element or the cast-in-place element.

6A.4.5: For lateral force resisting systems of Seismic Performance Category C, D or E, the nonlinear action connections shall be of Connection Performance Category C with the anchorage specified in Sec. 6A.4.4 and with the stressed area at the connection interface for nominal strength calculations at least 30 percent of the cross-sectional area of the element measured at a distance equal to the section's largest dimension from that interface. The stressed area for principal reinforcement stressed in tension or shear shall be the same as that defined in Sec. 10.6.4 of Ref. 6A-1.

6A.5 CONNECTION DESIGN REQUIREMENTS:

6A.5.1: Connections that are nonlinear action locations shall satisfy the following design requirements:

1. The probable strength, S_{pr}, of the connector shall be determined using a ϕ value of unity and a steel stress of at least $1.25f_y$.

2. The connector shall be anchored either side of the interface for capacities at least 1.6 times the S_{pr} value for that connector.

6A.5.2: Connectors that are part of the lateral load resisting path and intended to remain elastic while Connection Performance Category B or C connectors undergo nonlinear actions shall have a strength, S_n, at least 1.5 times the load calculated as acting on them when the nonlinear action of the building's structural system is fully developed.

6A.5.3: Connectors that are nonlinear action locations shall be proportioned so that they provide significant resistance only for the direction in which their capacity is intended to be utilized.

6A.5.4: Particular attention shall be given to grouting and welding requirements that shall permit quality control inspection and testing and make allowance for varying tolerances, material properties, and site conditions.

CHAPTER 7

COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS

7.1 SCOPE: Chapter 7* contains special requirements for the design and construction of composite members and systems combining structural steel and reinforced concrete in structures for which the design forces, generated by earthquake motions, have been determined according to the requirements of Chapter 2. Structures that do not meet the provisions in this chapter are permitted if substantiating evidence based on tests and analyses is provided that demonstrates that the structure has adequate strength, toughness, and seismic resistance for the intended purpose.

Except where modified by the provisions of this chapter, the provisions of Chapter 5, Steel Structure Design Requirements, and Chapter 6, Reinforced Concrete Structure Design Requirements, shall apply to this chapter where applicable.

7.2 REFERENCE DOCUMENTS: The quality and testing of composite steel-concrete materials and their design and construction to resist seismic forces shall conform to the relevant requirements of the following references except as modified by the provisions of this chapter.

Ref. 7-1 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (LRFD), American Institute of Steel Construction (AISC), 1993

Ref. 7-2 *Building Code Requirements for Reinforced Concrete*, American Concrete Institute, ACI-318-89 (Revised 1992), excluding Appendix A

Ref. 7-3 *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction (AISC), June 15, 1992, excluding Part II

Ref. 7-4 *Load and Resistance Factor Design Specification for Cold-formed Steel Structural Members,"* American Iron and Steel Institute (AISI), March 16, 1991 Edition

Ref. 7-5 *Standard for the Structural Design of Composite Slabs*, ASCE 3-91

* This chapter introduces into the *Provisions* seismic design requirements for composite steel-concrete structures. Due to a lack of design experience with certain types of composite systems under earthquake conditions, many of the recommendations are of a conservative and qualitative nature. Users of the *Provisions* are strongly encouraged to review the underlying source documents and research papers referenced in the Chapter 7 *Commentary*.

7.3 DEFINITIONS AND SYMBOLS:

7.3.1 DEFINITIONS:

Boundary Members: Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement and/or steel structural members.

Collector Elements: Members that serve to transfer forces between floor diaphragms and members of the lateral-force-resisting system.

Composite Beam: A steel beam either fully encased in concrete or an unencased steel beam made to act integrally with a concrete or composite slab using shear connectors.

Composite Brace: A steel brace fabricated from rolled or built-up structural steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete.

Composite Column: A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete where the structural steel portion accounts for at least 4 percent of the gross column area.

Composite Slab: Slab system consisting of a concrete slab and deformed metal deck where the two act compositely in flexure and shear.

Composite Shear Walls: Walls consisting of steel plates with concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel panel.

Coupling Beam: A steel or composite beam that is used to connect adjacent concrete wall piers to make them act together as a unit to resist lateral loads.

Encased Composite Beam : A steel beam totally encased in concrete cast integrally with the slab where full composite action is provided by bond between the steel and concrete without additional anchorage.

Encased Shapes: Structural steel members that are encased in structural concrete.

Face Bearing Plates: Stiffener plates that are attached to steel beams embedded in concrete walls or columns. The plates are located at the face of the concrete to provide confinement and transfer forces to the concrete through direct bearing.

Filled Tubes: Structural steel tubes or pipes that are filled with structural concrete.

Fully Composite Beam: A composite beam where the shear connectors are provided in sufficient numbers to develop the nominal plastic flexural strength of the composite section.

Load Carrying Bars: Reinforcing bars in composite members that are designed and detailed to resist calculated forces.

Partially Composite Beam: An unencased composite beam where the number and strength of shear connectors governs the flexural strength of the composite section.

Partially Restrained Composite Connection: Partially restrained connections as defined in Ref. 7-1 between partially or fully composite beams to steel columns where bending resistance is provided by a couple consisting of steel reinforcing bars in the slab and steel seat angles. The connections typically are designed to transmit moments less than the full capacity of the beams and columns.

Restraining Bars: Reinforcing bars in composite members that are provided primarily to facilitate erection of the reinforcement and to provide anchorage for stirrups or ties and are not designed to carry calculated forces. Generally, such bars are not spliced to be continuous.

Strength, Nominal: Strength of a member or cross section calculated in accordance with the provisions and assumptions of the strength design methods of these provisions (or the referenced standards) before application of any strength reduction factors.

Strength, Design: Nominal strength multiplied by a resistance factor, ϕ.

Strength, Required: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by these provisions.

7.3.2 SYMBOLS:

A_s = cross sectional area of structural steel elements in composite members.

A_s/A_g = ratio of cross sectional area of structural steel portion to the gross area of a composite column (see Ref. 7-1).

A_{sc} = area of encased steel shape on the compression side of the plastic neutral axis in a composite column.

A_{st} = area of encased steel shape on the tension side of the plastic neutral axis in a composite column.

A_{sp} = horizontal area of the steel plate in composite shear wall (Eq. 7.4.7.1.1).

b = flange width of structural steel tubing.

b_w = effective width for shear strength calculation of reinforced concrete portion of composite column.

b/t	=	width to thickness ratio of either unstiffened or stiffened elements, as applicable, of steel members under compression stresses (see Ref. 7-1).
d	=	depth of steel beam (Eq. 7.5.2.2).
D	=	the effect of dead load (Chapter 2).
D/t	=	diameter to wall thickness ratio of steel pipes used for concrete filled composite columns (see Ref. 7-1).
E	=	the load effect of seismic (earthquake-induced) forces (Chapter 2).
E_s	=	elastic modulus of structural steel.
F_y	=	specified yield strength of structural steel.
F_{yh}	=	specified yield strength of reinforcing bar ties.
f_c'	=	specified compressive strength of concrete used in design.
h	=	the cross sectional dimension of reinforced concrete or composite columns.
h_c	=	the cross sectional dimension of the confined core region in composite columns measured center-to-center of the tie reinforcement.
h/t_w	=	height to thickness ratio of web elements of steel members (see Ref. 7-1).
P_n	=	nominal axial strength of column.
P_u	=	required axial design strength of column.
R	=	the response modification coefficient as given in Table 2.2.2.
V_c	=	portion of nominal shear strength provided by the concrete in reinforced concrete or composite columns (see Ref. 7-2).
V_s	=	shear yield strength of steel plate in composite shear wall (Eq. 7.4.7.1.1).
Y_{con}	=	distance from top of steel beam to top of concrete slab or encasement (Eq. 7.5.2.2).
ϕ_c	=	resistance factor for composite columns (= 0.85).
ρ_v	=	ratio of vertical or horizontal reinforcement in walls (see Chapter 21 of Ref. 7-2).

7.4 COMPOSITE SYSTEMS: The use and height restrictions of composite building systems shall be as specified in Table 2.2.2. Except as noted in this section, structural steel and reinforced concrete members in composite systems shall satisfy the provisions of Chapters 5 and 6, respectively. Composite members shall meet the provisions of Sec. 7.5. Connections between steel, concrete, and composite members shall meet the provisions of Sec. 7.6.

7.4.1 COMPOSITE PARTIALLY RESTRAINED FRAMES (C-PRF): The provisions in this section apply to frames consisting of structural steel columns and composite beams, connected with partially restrained connections meeting the requirements of Type PR construction as defined in Ref. 7-1.

7.4.1.1 Limitations:

7.4.1.1.1: The effect of the connection flexibility shall be accounted for in determining the strength and drift of C-PRF.

7.4.1.1.2: The height restrictions in Table 2.2.2 for C-PRF may be exceeded if the strength and ductility of the structure are demonstrated through nonlinear analysis that includes both the geometric and connection nonlinearities.

7.4.1.2 Columns: Steel columns shall be designed according to Ref. 7-1 with the effect of the semi-rigid connections considered in evaluating the stability of individual columns and the frame. Required design strengths for columns shall include moments transferred into the columns through the semi-rigid connections. In Seismic Performance Category D, steel columns also shall meet the requirements of Ref. 7-3, Sec. 6.

7.4.1.3 Composite Beams: Composite beams shall be designed according to the requirements of Sec. 7.5.2. For analysis, beam stiffness shall be determined using an effective moment of inertia of the composite section.

7.4.1.4 Partially Restrained Connection: Beam-column moment connections shall be designed for the factored loads calculated according to the provisions in Chapter 2 and considering the connection flexibility.

7.4.2 COMPOSITE ORDINARY MOMENT FRAMES (C-OMF): The provisions in this section apply to moment resisting frames consisting of either steel, reinforced concrete, or composite columns and either steel or composite beams.

7.4.2.1 Columns: Steel columns shall meet the requirements for ordinary moment frames in Chapter 5.

In Seismic Performance Categories A, B and C, reinforced concrete columns shall meet the requirements for intermediate moment frames in Chapter 6. In Seismic Performance Categories D and E, reinforced concrete columns shall meet the requirements for special moment frames in Chapter 6.

Composite columns shall meet the requirements of Sec. 7.5.

7.4.2.2 Beams: Steel beams shall meet the requirements for ordinary moment frames in Chapter 5. Composite beams shall meet the requirements of Sec. 7.5.2.

7.4.2.3 Moment Connections: In Seismic Performance Categories A, B and C, beam-column moment connections shall be designed for the factored loads calculated according to Chapter 2. In Seismic Performance Categories D and E, moment connections also shall be designed to meet one of the following requirements:

a. The connection design strength shall meet or exceed the forces associated with plastic hinging of the beams adjacent to the connection,

b. The connection design strength shall meet or exceed the forces calculated using Eq. 2.2.6-3 and 2.2.6-4, or

c. The connections are demonstrated by cyclic tests to have adequate rotation capacity at a story drift calculated from an elastic analysis at a horizontal load of $2R/5 \times E$ where $2R/5$ is greater than or equal to 1.0.

7.4.3 COMPOSITE SPECIAL MOMENT FRAMES (C-SMF): The provisions in this section apply to moment resisting frames consisting of either steel, reinforced concrete or composite columns and either steel or composite beams. Members and connections in C-SMF shall be designed to provide ductility and toughness that permit the use of lower seismic design forces than C-OMF.

7.4.3.1 Columns: Steel columns shall meet the requirements for special moment frames in Chapter 5. Reinforced concrete columns shall meet all requirements for special moment frames in Chapter 6. Composite columns shall meet the requirements of Sec. 7.5.

7.4.3.2 Beams: Steel beams shall meet the requirements for special moment frames in Chapter 5. Composite beams shall meet the requirements of Sec. 7.5.2. Steel or composite trusses are not permitted for use as flexural members to resist lateral loads in composite SMF, unless it is shown by tests and analysis that the particular system provides adequate ductility and energy dissipation capacity.

7.4.3.3 Moment Connections: Moment connection design strengths shall meet or exceed the flexural and shear forces associated with plastic hinging of the beams adjacent to the connection.

7.4.3.4 Column-Beam Moment Ratio: When $P_u > 0.1P_n$ in reinforced concrete or composite columns, one of the following provisions shall be met:

a. The flexural strength of the columns shall meet the requirements of Sec. 21.4.2.2 of Ref. 7-2 or

b. For reinforced concrete columns the provisions of Sec. 21.4.2.3 of Ref. 7-2 shall be met, and for encased composite columns the hoop reinforcement specified in Sec. 7.5.3.8.3(b) shall extend over the full column height.

The provisions of Sec. 8.6 of Ref. 7-3 shall be met for steel columns.

7.4.4 COMPOSITE CONCENTRICALLY BRACED FRAMES (C-CBF): The provisions in this section apply to braced systems composed of concentrically connected members. Minor eccentricities are acceptable if accounted for in the design. Columns may be either reinforced concrete or composite. Beams and braces may be either steel or composite.

7.4.4.1 Limitations: C-CBF shall meet all requirements for steel CBF in Chapter 5 except as modified in this section.

7.4.4.2 Columns: Reinforced concrete columns shall meet all requirements for intermediate moment frames in Chapter 6. In Seismic Performance Categories D and E reinforcement in concrete columns also shall meet the requirements of Sec. 21.6.2.3 of Ref. 7-2.

Composite columns shall meet the requirements of Sec. 7.5.

7.4.4.3 Beams: Composite beams shall meet the requirements of Sec. 7.5.2.

7.4.4.4 Braces: Composite braces shall meet the requirements for composite columns in Sec. 7.5. Unless evidence is provided to justify higher strengths, composite braces shall be designed in tension as steel braces according to the requirements for steel CBF in Chapter 5.

7.4.4.5 Bracing Connections: In addition to the requirements for steel CBF in Chapter 5, bracing connections shall meet all requirements of Sec. 7.6.

7.4.5 COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF): The provisions in this section apply to braced systems where at least one end of each brace intersects a link beam at a prescribed eccentricity from the intersection of either the column and link-beam centerlines or the adjacent brace and link-beam centerlines. Eccentrically braced frames of composite steel-concrete shall be designed so that under earthquake loading, yielding will be primarily shear yielding and will occur in the links. The diagonal braces, columns and 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. Columns may be either reinforced concrete or composite. Braces shall be structural steel. Links shall be as specified in 7.4.5.3.

7.4.5.1 Limitations: C-EBF shall meet all requirements for steel EBF as specified in Chapter 5 except as modified in this section.

7.4.5.2 Columns: Reinforced concrete columns shall meet the requirements for intermediate moment frames in Chapter 6. In Seismic Performance Categories D and E or where shear links are directly adjacent to the column, transverse reinforcement for concrete columns also shall meet the requirements of Sec. 21.6.2.3 of Ref. 7-2.

Composite columns shall meet the requirements of Sec. 7.5. Additionally, where shear links are directly adjacent to the column, transverse reinforcement in composite columns shall meet the requirements of Sec. 7.5.3.8.3.

All steel columns shall satisfy the strength requirements of Sec. 10.8 of Ref. 7-3.

7.4.5.3 Link Beams: Link portions of beams shall be unencased and shall meet the requirements for links in steel EBF in Chapter 5. Portions of the beams outside the link region may be encased in concrete. Link beams may be connected to the floor slab with shear connectors along all or any portion of their length provided that the composite action is considered in calculating the maximum nominal strength of the link.

7.4.5.4 Braces: Steel braces shall meet or exceed the requirements for steel EBF braces in Chapter 5.

7.4.5.5 Connections: In addition to the requirements for steel EBF connections in Chapter 5, connections shall meet requirements of Sec. 7.6.

7.4.6 RC WALLS COMPOSITE WITH STEEL ELEMENTS: The provisions in this section apply where reinforced concrete walls are composite with steel elements in any of the following ways:

a. Reinforced concrete walls are used as infill panels in structural steel frames and the steel members are either non-encased or encased,

b. Encased shapes are used to reinforce boundary members in reinforced concrete shear walls, or

c. Steel coupling beams are used to connect two or more reinforced concrete walls.

7.4.6.1 Limitations Except as modified in this section, reinforced concrete walls shall meet the requirements of Chapter 6.

7.4.6.2 Boundary Elements:

7.4.6.2.1 Steel Shapes: Where reinforced concrete infill panels are used and unencased steel shapes function as boundary elements, the steel columns shall meet the design provisions of Ref. 7-1 and Sec. 5 and 6 of Ref. 7-3. Element forces in the steel boundary members shall be calculated following the provisions in Sec. 21.6.5.3 of Ref. 7-2 where it shall be assumed that the reinforced concrete panel only resists shear forces and the steel columns carry 100 percent of the vertical forces due to gravity loads and overturning in the wall.

7.4.6.2.2 Encased Steel Shapes: Where steel shapes are fully encased to form composite boundary elements, boundary element forces shall be calculated following the provisions in Sec. 21.6.5.3 of Ref. 7-2. For purposes of stress calculations, the composite wall may be transformed into an equivalent concrete section using elastic material properties. Encased boundary elements shall meet the requirements of Sec. 7.5, 7.5.3.3, 7.5.3.4, and 7.5.3.6 of this document. In Seismic Performance Categories D and E, boundary elements also shall meet the boundary element requirements of Sec. 21.6.5 of Ref. 7-2 and Sec. 7.5.3.7 of this document. Transverse reinforcement for confinement of the composite boundary element shall extend a distance of $2h$ into the wall where h is the overall depth of the boundary member in the plane of the wall.

7.4.6.2.3 Shear Connectors: Headed studs or welded reinforcing bar anchors shall be provided to transfer vertical shear forces between the reinforced concrete and structural steel members. The nominal strengths of headed studs may be calculated following the provisions of Ref. 7-1. For connection to unencased steel shapes in Seismic Performance Categories C, D and E, the nominal strengths of headed studs shall be reduced by 25 percent from those specified in Ref. 7-1 and the nominal strength of welded reinforcing bar anchors shall be reduced by 25 percent from their static yield strength.

7.4.6.3 Coupling Beams: Steel beams may be used to couple adjacent concrete walls or columns and shall meet the requirements of this section.

7.4.6.3.1: Coupling beams shall satisfy the provisions of the following sections of Ref. 7-3: 10.2.a through Sec. 10.2.f, Sec. 10.3.b where the coupling rotation shall be assumed as 0.09 radians, and Sec. 10.3.c. Full depth web stiffeners (face bearing plates) also shall be provided on both sides of the coupling beams at the face of the concrete wall. These stiffeners shall meet the detailing requirements of Sec. 10.3.a in Ref. 7-3.

7.4.6.3.2: Embedment length of coupling beams in concrete walls shall be sufficient to develop the maximum possible combination of moment and shear force that can be generated by the nominal strength of the coupling beam. Embedment length shall be considered to begin inside the first layer of confinement reinforcement in the wall boundary member. Connection strength for transfer of loads between coupling beams and concrete walls shall meet the requirements of Sec. 7.6.

7.4.6.3.3: Vertical wall reinforcement with an axial strength equal to the calculated nominal shear strength of the coupling beam shall be placed over the embedment length of the link beam with two thirds of the steel located over the first half of the embedment length. This wall reinforcing shall extend a distance of at least one tension development length above and below the flanges of the link beams. This vertical reinforcement shall be confined by transverse reinforcement at least equivalent to that required in Chapter 6 for boundary elements in reinforced concrete walls. The reinforcement required by this section is only a minimum amount which may be satisfied by vertical reinforcement provided to satisfy other requirements such as for reinforcing the wall boundary elements.

7.4.7 COMPOSITE SHEAR WALLS: The provisions in this section apply to structural walls consisting of steel plates with concrete encasement on one or both sides of the plate and steel or composite boundary elements.

7.4.7.1 Wall Element:

7.4.7.1.1 Calculation of Shear Strength: Where the stiffening requirements of Sec. 7.4.7.1.2 are met, the nominal shear strength of the wall shall be calculated as the following:

$$V_s = \frac{A_{sp} F_y}{\sqrt{3}} \tag{7.4.7.1.1}$$

where

V_s = the shear yield strength of the steel plate,

A_{sp} = the horizontal area of the steel plate, and

F_y = the specified yield strength of the plate.

For cases where the stiffening requirements of Sec. 7.4.7.1.2 are not met, the shear strength of the steel plate shall be calculated following the provisions for steel shear members in Chapter 5 considering buckling of the steel plate. Shear strength of the reinforced concrete shall not be included in the strength calculation for the composite wall.

7.4.7.1.2 Concrete Stiffening: The steel plate may be considered to be sufficiently stiffened by concrete if an elastic plate buckling analysis shows that the composite wall can resist an applied shear force equal to its nominal shear strength. Additionally, the concrete thickness shall be a minimum of 4 in. (102 mm) when concrete is provided on both sides of the steel plate and a minimum of 8 in. (203 mm) when concrete is provided on one side of the steel plate. Headed studs or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and concrete. Reinforcement equal to that specified in Sec. 14.3 of Ref. 7-2 shall be provided in the concrete encasement. In Seismic Performance Categories D and E, the reinforcement ratio in either the horizontal or vertical direction, ρ_v, shall not be less than 0.0025 and the maximum spacing between bars shall not exceed 18 in. (457 mm).

7.4.7.1.3 Shear Transfer: The steel plate shall be continuously connected on all edges to steel boundary elements by welds and/or slip critical bolts to develop a shear force at least equal to the shear yield strength of the plate. The design strength of welded and bolted connectors shall meet the provisions of Ref. 7-1.

7.4.7.2 Boundary Members: Steel and composite boundary members shall be designed following the provisions in Sec. 7.4.6.2.

7.4.7.3 Openings in Composite Walls: Boundary members shall be provided around openings as required by analysis.

7.5 COMPOSITE MEMBERS: The design of composite members subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined by the provisions of Ref. 7-1 through 7.5 except as modified in this section.

Structural steel in composite members shall meet the requirements of Ref. 7-1, Sec. A3.1. Additionally, for special moment frames and for all structures in Seismic Performance Categories D and E, structural steel in composite members shall meet the requirements of Ref. 7-3, Sec. 5.

Reinforcing steel in composite members shall meet the requirements of Ref. 7-2, Sec. 3.5.1, 3.5.2, and 3.5.3. Additionally, in Seismic Performance Categories C, D and E and in special moment frames, reinforcing steel shall meet the requirements of Ref. 7-2, Sec. 21.2.5 and 21.2.6.

Concrete in composite members shall have a specified compressive strength not less than 3 ksi (20.7 MPa). Additionally, in composite members and in composite columns in Seismic Performance Categories C, D and E, special moment resisting frames, the design compressive strength of normal weight concrete shall not exceed 10 ksi (68.9 MPa) and for lightweight concrete shall not exceed 4 ksi (27.6 MPa). Concrete with higher design compressive strengths may be allowed if demonstrated by experimental evidence that structural members made with that concrete provide sufficient strength and toughness for the intended purpose.

7.5.1 COMPOSITE SLABS: Composite and noncomposite floor slabs shall be designed to meet the requirements of Chapter 2. Steel deck diaphragms and concrete slab diaphragms shall be designed to meet the provisions of Chapters 5 and 6, respectively. Where concrete slab on steel deck diaphragms are used to resist seismic forces the requirements of this section shall be met.

7.5.1.1 Shear Fasteners: Provisions shall be made to transfer forces between the composite slab and the boundary members and collector elements of the diaphragm.

7.5.1.2 Shear Strength: The in-plane design shear strength of the composite diaphragms shall be calculated on the basis of established design procedures or tests of concrete filled diaphragms; or conservatively, on the basis of only the reinforced concrete slab above the top of the ribs of the metal deck. In the latter case, the nominal in-plane strength of the reinforced concrete slab shall meet the provisions of Chapter 6.

7.5.2 COMPOSITE BEAMS: The provisions in this section apply only to those composite beams that are part of the primary lateral load resisting system in the structure. Except as noted below, the design provisions in Chapter I of Ref. 7-1 shall apply.

7.5.2.1 Additional Requirements for Special Moment Frames: Design provisions for special moment frames in Sec. 8.8 of Ref. 7-3 shall apply except as modified in Sec. 7.5.2.2 through 7.5.2.3 of this chapter. In Seismic Performance Categories D and E, the provisions of Sec. 8.4 of Ref. 7-3 also apply.

7.5.2.2 Plastic Stress Distribution: The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed:

$$\frac{Y_{con} + d}{1 + \left(\frac{1{,}700 F_y}{E_s}\right)} \tag{7.5.2.2}$$

where

Y_{con} = the distance from the top of the steel beam to the top of concrete,

d = the depth of the steel beam, and

F_y = the specified yield strength of the steel beam, and

E_s = the elastic modulus of the steel beam.

7.5.2.3 Width-Thickness Ratios: Compression elements that are fully encased by a reinforced concrete cover of at least 2 in. (51 mm) do not need to meet the width-thickness ratios of Sec. 8.4 in Ref. 7-3 provided that concrete is confined by hoop reinforcement in regions where plastic hinges are expected to occur under seismic loading. Hoop reinforcement shall meet the requirements of Sec. 21.3.3 in Ref. 7-2.

7.5.3 ENCASED COMPOSITE COLUMNS: Columns consisting of structural concrete encased steel sections where the structural steel area comprises at least 4 percent of the total composite column cross section and meeting the additional limitations in Sec. I 2.1 of Ref. 7-1 shall be designed according to provisions in Chapter I of Ref. 7-1 and the provisions in this section. Columns with encased shapes that do not meet these criteria shall be designed according to the provisions of Chapter 6 for reinforced concrete columns.

7.5.3.1 Shear Strength: The design shear strength of the column shall be determined by the shear strength of the structural shape plus the shear strength of tie reinforcement in the reinforced concrete encasement. The shear strength of the steel shape shall be calculated following the provisions of Ref. 7-1, Sec. F2. The shear strength of the tie reinforcement shall be calculated following the provisions of Ref. 7-2, Sec. 11.5.6.2 through 11.5.6.8. For Sec. 11.5.6.4 and 11.5.6.8, the dimension b_w shall be equal to the width of the concrete cross section minus the width of the structural shape measured perpendicular to the direction of shear.

7.5.3.2 Shear Connectors: Where a composite column is designed so that the steel shape and concrete share the applied loads, shear connectors shall be provided to meet the following requirements:

a. If external members are framed directly to the steel shape, provide shear connectors to transfer the sum of all applied forces along the axis of the member that are not able to be carried by the steel section alone.

b. If external members are framed directly to the concrete in direct bearing or shear, provide shear connectors to transfer the sum of all applied forces along the axis of the member that are not able to be carried by the reinforced concrete alone.

c. Where $P_u/\phi_c P_n \leq 0.3$ and a composite column is subjected to predominantly flexure, provide shear connectors along the axis of the embedded shape from the point of inflection to the point of maximum moment designed for a force equal to $F_y(A_{st} - A_{sc})$ where F_y is the yield strength of the structural steel and A_{st} and A_{sc} are the areas of embedded steel shape on the tensile and compression side of the plastic neutral axis, respectively.

d. The maximum spacing of shear connectors shall be 32 in. (813 mm) with attachment along the outside flange faces of the embedded shape.

7.5.3.3 Transverse Reinforcement: The maximum spacing of transverse ties shall be the smaller of the following:

a. 1/2 the least dimension of the section,

b. 16 diameters of the longitudinal bars, or

c. 48 tie bar diameters.

Ties shall be located vertically not more than 1/2 a tie spacing above the top of footing or lowest beam or slab in any story, and shall be spaced as provided herein to not more than 1/2 a tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter not less than 1/50 times the greatest side dimension of the composite members, except that ties shall not be smaller than No. 3 (10 mm diameter) bars and are not required to be larger than No. 5 (16 mm diameter) bars. Welded wire fabric of equivalent area is permitted as transverse reinforcement in Seismic Performance Categories A and B.

7.5.3.4 Longitudinal Reinforcement: All load carrying bars shall meet the detailing and splice provisions of Ref. 7-2, Sec. 7.8.1 and 12.17. As a minimum, load carrying bars must be provided at every corner of a rectangular cross section. Other load carrying or restraining longitudinal bars shall not be spaced farther apart than one-half of the least side dimension of the composite member.

7.5.3.5 Steel Core: Splices and end bearing details for the encased steel shape shall meet the requirements of Sec. 7.8.2 of Ref. 7-2 and Ref. 7-1.

7.5.3.6 Additional Requirements in Seismic Performance Category C: Concrete encased composite columns in Seismic Performance Category C shall meet the requirements in this section in addition to those in Sec. 7.5.3.1 through 7.5.3.5.

Maximum spacing of transverse bars at the top and bottom of a story shall not exceed the smaller of the following:

a. 1/2 the least dimension of the section,
b. 8 bar diameters of the longitudinal bars,
c. 24 tie bar diameters, or
d. 12 in. (305 mm).

These spacings shall be maintained at least over a vertical distance equal to the greater of the following lengths measured from each joint face and on both sides of any section where flexural yielding is likely to occur:

a. 1/6 the vertical clear height of the column,

b. the maximum cross sectional dimension, and

c. 18 in. (457 mm).

Tie spacing over the rest of the column length shall not exceed twice the spacing defined above.

7.5.3.7 Additional Requirements in Seismic Performance Categories D and E: Encased columns in Seismic Performance Categories D and E shall meet the requirements in this section in addition to those in Sec. 7.5.3.1 through 7.5.3.6.

7.5.3.7.1 Columns: Seismic design forces for columns shall be calculated using Eq. 2.2.6-3 and 2.2.6-4.

7.5.3.7.2 Transverse Reinforcement: Ties shall be provided at the top and bottom of a column that meet the requirements of Sec. 7.5.3.6 and this section. The requirements of this section need not apply if the nominal strength of the encased steel section alone is greater than 1.0*D* plus 0.5*L* where *D* and *L* are defined in Chapter 2.

The minimum area of reinforcement, A_{sh}, shall be at least equal to the following:

$$A_{sh} = 0.09 h_c s \left(1 - \frac{F_y A_s}{P_n}\right)\left(\frac{f_c'}{F_{yh}}\right) \tag{7.5.3.7.2}$$

where

h_c = the cross-sectional dimension of the confined core region measured center-to-center of the tie reinforcement,

F_y = the specified yield strength of the structural steel core,

A_s = the cross sectional area of the structural steel core,

P_n = the nominal compressive axial strength of the composite column calculated according to Ref. 7-1,

f_c' = the specified concrete compressive strength, and

F_{yh} = the specified yield strength of the ties.

7.5.3.7.3 Longitudinal Bars: Design of load carrying longitudinal reinforcing bars shall meet the requirements of Ref. 7-2, Sec. 21.4.3.

7.5.3.7.4 Steel Core: Splices of the encased structural steel core shall meet the requirements of Ref. 7-3, Sec. 6.2.

7.5.3.7.5 Columns Supporting Discontinuous Walls: Composite columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as specified in Sec. 7.5.3.6 over the full height beneath the level at which the discontinuity occurs if the axial compression exceeds $0.10P_n$ (where P_n is the nominal compressive axial strength of the composite column). Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the encased shape and longitudinal reinforcing bars.

7.5.3.7.6 Columns Supported by Walls or Footings: Where the column terminates on a wall, the transverse reinforcement as specified in Sec. 7.5.3.5 shall extend into the wall for at least the length required to develop full yielding in the encased shape and longitudinal reinforcing bars. Where the column terminates on a footing or mat, the transverse reinforcement as specified in Sec. 7.5.3.5 shall extend into the footing or mat at least 12 in. (305 mm).

7.5.3.8 Additional Requirements in Special Moment Frames: Encased columns used in special moment frames in all Seismic Performance Categories shall meet the requirements in this section in addition to those in Sec. 7.5.3.1 through 7.5.3.7.

7.5.3.8.1 Strong Column/Weak Beam: The strong-column/weak-beam design requirements of Sec. 7.4.3.4 shall be satisfied to limit plastic hinge formations in the columns. Column bases shall be detailed to sustain inelastic flexural hinging.

7.5.3.8.2 Shear Strength: The minimum required nominal shear strength of the column shall meet the provisions of Sec. 21.4.5.1 of Ref. 7-2.

7.5.3.8.3 Transverse Reinforcement: Transverse reinforcement shall be detailed as hoop reinforcement (Chapter 21 of Ref. 7-2) and shall meet the more severe of the requirements of Sec. 7.5.3.3, 7.5.3.6 and 7.5.3.7 and the following:

a. The maximum spacing of transverse reinforcement along the entire length of the column shall not exceed 6 diameters of the longitudinal load carrying bars or 6 in. (152 mm).

b. At the top and bottom of the column over the region specified in Sec. 7.5.3.6 the maximum spacing of transverse reinforcement shall not exceed 1/4 of the minimum member dimension or 4 in. (102 mm). In these regions, crossties, legs of overlapping hoops, or other confining reinforcement shall not be spaced more than 14 in. (356 mm) on center in the transverse direction.

7.5.4 FILLED COMPOSITE COLUMNS: Columns consisting of concrete filled steel tubes or pipes where the structural steel area comprises at least 4 percent of the total composite column cross section and meeting the additional limitations of Sec. I 2.1 in Ref. 7-1 shall be designed according to provisions in Chapter I of Ref. 7-1 and the provisions in this section.

7.5.4.1 Shear Strength: The shear strength of column shall be calculated as the strength of the steel section alone.

7.5.4.2 Additional Requirements in Seismic Performance Categories D and E: Filled columns in Seismic Performance Categories D and E shall meet the requirements in this section in addition to those in Sec. 7.5.4.1.

7.5.4.2.1 Columns: Seismic design forces in columns shall be calculated using Eq. 2.2.6-3 and 2.2.6-4.

7.5.4.2.2 Steel Tube: Splices of the structural steel tube or pipe shall meet the requirements of Ref. 7-3, Sec. 6.2.

7.5.4.3 Additional Requirements for Special Moment Frames: Filled columns used in special moment frames in all Seismic Performance Categories shall meet the requirements in this section in addition to those in Sec. 7.5.4.1 and 7.5.4.2.

7.5.4.3.1 Shear Strength: The minimum required shear strength of the column shall meet the provisions of Sec. 21.4.5.1 of Ref. 7-2.

7.5.4.3.2 Strong Column/Weak Beam: The strong-column/weak-beam design requirements of Sec. 7.4.3.4 shall be satisfied to limit plastic hinge formations in the columns. Column bases shall be detailed to sustain inelastic flexural hinging.

7.5.4.3.3 Structural Steel Pipe and Tubing: The minimum wall thickness of structural steel pipe or tubing filled with concrete shall be equal to $b\sqrt{(F_y/2E_s)}$ for each face of width b in rectangular sections and $D\sqrt{(F_y/5E_s)}$ for circular sections of outside diameter D.

7.6 COMPOSITE CONNECTIONS: The requirements in this section apply to connections in structures with composite or dual steel-concrete systems where the interaction of structural steel and reinforced concrete components is relied upon for transfer of seismic force between members. Where the interaction between structural steel and reinforced concrete is not required for the transfer of seismic forces between members, the connections shall be designed for seismic forces according to the provisions in Chapter 5 or Chapter 6.

Composite connections shall be demonstrated to have strength, ductility, and toughness at least equal to those for similar structural steel or reinforced concrete connections that meet the provisions of Chapter 5 and Chapter 6. Methods for calculating the connection strength shall meet the provisions in this section.

7.6.1 GENERAL REQUIREMENTS: All connections in the structure shall have adequate deformation capacity to resist their critical factored design loads under the design story drifts calculated according to the requirements of Chapter 2. Additionally, connections that are required for lateral stability under seismic forces shall meet the provisions of this chapter.

Connections shall be designed for the required strengths specified in Sec. 7.4 based on the specific system in which the connection is used. Where minimum connection design forces are based on the flexural and/or axial capacity of connected members, the forces shall be determined based on specified nominal material strengths and nominal dimensions of the members. In such cases, the minimum connection strength calculations shall account for effects that may increase the ultimate strengths of members above the nominal strengths calculated for design of the member.

7.6.2 STRENGTH DESIGN CRITERIA: Calculated connection strengths shall be based on rational models that satisfy equilibrium of internal forces and strength limitations of component materials and elements based on potential failure modes. Unless the connection strength is determined by tests and analysis, connection design models shall follow the criteria presented in Sec. 7.6.2.1 to 7.6.2.5.

7.6.2.1 Force Transfer Between Structural Steel and Concrete: Force transfer between structural steel and concrete shall only be considered to occur through direct bearing and/or shear friction. Force transfer shall be calculated based only on direct bearing forces and/or clamping forces provided by reinforcement, shear studs, or other mechanical devices. Bond between steel and concrete is not to be considered as a connection force transfer mechanism.

7.6.2.2 Structural Steel Elements: The design strength of steel components of connections shall not exceed those prescribed in Ref. 7-1 and 7-3. Steel elements are permitted to be considered to be braced against out-of-plane buckling effects when they are encased in confined concrete. Face bearing plates that consist of stiffeners between the flanges of steel beams are required where beams are embedded into concrete columns or walls.

7.6.2.3 Shear Friction and Bearing Stresses in Concrete: Ultimate bearing and shear friction design strengths shall not exceed those prescribed in Chapters 10 and 11 of Ref. 7-2. In Seismic Performance Categories D and E, unless higher values are substantiated by cyclic tests, the bearing and friction design strengths calculated on the basis of Ref. 7-2 shall be reduced by 25 percent.

7.6.2.4 Panel Zones: The panel zone shear strength may be calculated as the sum of the strengths of the structural steel and reinforced concrete shear elements where each is calculated following the provisions of Sec. 8 of Ref. 7-3 and Sec. 21.5 of Ref. 7-2.

7.6.2.5 Reinforcing Bar Detailing Provisions: Reinforcing bars shall be provided to resist all calculated tensile forces in reinforced concrete components of the connections. Transverse reinforcement also shall be designed to provide confinement of the concrete. All reinforcement shall be fully developed in tension or compression beyond the section where it is required for resisting forces. Development lengths shall be determined following the provisions of Chapter 12 of Ref. 7-2. In Seismic Performance Categories D and E, development lengths also shall meet the requirements of Sec. 21.5.4 of Ref. 7-2.

7.6.2.5.1 Slab Reinforcement in Connection Region: Where the slab is used to transfer horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the calculated in-plane tensile forces at all critical sections in the slab including connections to collector beams, columns, braces and walls.

7.6.2.5.2 Transverse Reinforcement in Columns or Walls Near Joint: For connections to reinforced concrete or encased composite columns, transverse hoop reinforcement shall be provided in the joint region according to the provisions of Sec. 21.5 of Ref. 7-2 with the following modifications:

a. Steel members framing into the connection shall be considered to provide confinement over a width equal to that of face bearing stiffener plates welded to the beams between the flanges and

b. In Seismic Performance Categories A, B and C and for ordinary composite moment frames in Seismic Performance Categories D and E, lap splices may be used for perimeter ties where confinement of the splice is provided by face bearing plates or other means to prevent spalling of the concrete cover.

7.6.2.5.3 Longitudinal Reinforcement in Columns Near Joints: The longitudinal bar sizes and layout shall be detailed to minimize slippage of the bars through the joint due to high force transfer associated with the change in column moments over the height of the joint.

Chapter 8

MASONRY STRUCTURE DESIGN REQUIREMENTS

8.1 GENERAL:

8.1.1 SCOPE: The design and construction of reinforced and plain (unreinforced) masonry components and systems and the materials used therein shall comply with the requirements of this chapter.

8.1.2 REFERENCE DOCUMENTS: The designation and title of documents cited in this chapter are listed in this section. Compliance with specific provisions of these reference documents is mandatory where required by this chapter.

Design and Construction Standards:

Ref. 8-1 *Building Code Requirements for Masonry Structures*, ACI 530-92/ASCE 5-92/TMS 402-92

Ref. 8-2 *Specifications for Masonry Structures*, ACI 530.1-92/ASCE 6-92/TMS 602-92

Materials Standards:

Ref. 8-3 *Specification for Structural Steel*, ASTM A36-90

Ref. 8-4 *Specification for Steel Wire, Plain, for Concrete Reinforcement*, ASTM A82-90a

Ref. 8-5 *Specification for Steel Wire, Deformed, for Concrete Reinforcement*, ASTM A496-90a

Ref. 8-6 *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*, ASTM A615-90

Ref. 8-7 *Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM A616-90

Ref. 8-8 *Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement*, ASTM A617-90

Ref. 8-9 *Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement*, ASTM A706-90

Ref. 8-10 Requirements for Joint Reinforcement, Sec. 3.2 of Ref. 8-2

Ref. 8-11 *Specification for Structural Clay Load-Bearing Wall Tile*, ASTM C34-84(1991)

Ref. 8-12 *Specification for Concrete Building Brick*, ASTM C55-85

Ref. 8-13 *Specification for Building Brick (Solid Masonry Units Made from Clay or Shale)*, ASTM C62-91b

Ref. 8-14 *Test Methods of Sampling and Testing Brick and Structural Clay Tile*, ASTM C67-91

Ref. 8-15 *Specification for Calcium Silicate Face Brick (Sand-Lime Brick)*, ASTM C73-85(1989)

Ref. 8-16 *Specification for Hollow Load-Bearing Concrete Masonry Units*, ASTM C90-93

Ref. 8-17 *Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units*, ASTM C126-91

Ref. 8-18 *Test Methods of Sampling and Testing Concrete Masonry Units*, ASTM C140-91

Ref. 8-19 *Specification for Structural Clay Facing Tile*, ASTM C212-91

Ref. 8-20 *Specification for Facing Brick (Solid Masonry Units Made From Clay or Shale)*, ASTM C216-91c

Ref. 8-21 *Specification for Mortar for Unit Masonry*, ASTM C270-91a

Ref. 8-22 *Test Method for Drying Shrinkage of Concrete Block*, ASTM C426-70(1988)

Ref. 8-23 *Specification for Grout for Masonry*, ASTM C476-91

Ref. 8-24 *Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)*, ASTM C652-91c

Ref. 8-25 *Specification for Prefaced Concrete and Calcium Silicate Masonry Units*, ASTM C744-73(1985)

Ref. 8-26 *Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry*, ASTM C780-91

Ref. 8-27 *Specification for Elastomeric Joint Sealants*, ASTM C920-87

Ref. 8-28 *Method of Sampling and Testing Grout*, ASTM C1019-89a

Ref. 8-29 *Specification for Ready Mix Mortar*, ASTM C1142-90

Ref. 8-30 Test Method for Compressive Strength of Masonry, Sec. 1.5 of Ref. 8-1

Ref. 8-31 *Specification for Preformed Expansion Joint Filler for Concrete (Bituminous Type)*, ASTM D994-83

Ref. 8-32 *Specification for Flexible Cellular Materials - Sponge or Expanded Rubber*, ASTM D1056

Ref. 8-33 *Classification System for Rubber Products in Automotive Applications*, ASTM D2000

Ref. 8-34 *Specification for Nonrigid Vinyl Chloride Polymer and Copolymer Molding and Extrusion Compounds*, ASTM D2287-81(1988)

Ref. 8-35 *Specification for Strength of Anchors in Concrete and Masonry Elements*, ASTM E488

Ref. 8-36 *Structural Welding Code - Reinforcing Steel*, ANSI/AWS D1.4-92

8.1.3 DEFINITIONS:

Anchor: Metal rod, wire, bolt, or strap that secures masonry to its structural support.

Area:

Gross Cross-Sectional Area: The area delineated by the out-to-out specified dimensions 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 based on out-to-out specified dimensions.

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 Masonry: Multiwythe masonry members acting with composite action.

Connector: A mechanical device (including anchors, wall ties, and fasteners) for joining two or more pieces, parts, or members.

Dimension:

Actual Dimension: The measured dimension of a designated item (e.g., a designated masonry unit or wall).

Nominal Dimension: The specified dimension rounded to the nearest whole number.

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: For braced members, the effective height is the clear height between lateral supports and is used for calculating the slenderness ratio. The effective height for unbraced members is calculated in accordance with engineering mechanics.

Effective Period: Fundamental period of the building based on cracked stiffness.

Head Joint: Vertical mortar joint 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.

Plain Masonry: Masonry in which the tensile resistance of the masonry is taken into consideration and the effects of stresses in reinforcement are neglected.

Plastic Hinge: The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquake.

Reinforced Masonry: Masonry construction in which reinforcement acts 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: Required by contract documents.

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 (MPa).

Stack Bond: Stack bond is other than running bond. Usually, the placement of units is such that the head joints in successive courses are aligned vertically.

Stirrup: Shear reinforcement in a beam or flexural member.

Strength:

Design Strength: Nominal strength multiplied by a strength reduction factor.

Nominal Strength: Strength of a member or cross section calculated in accordance with these 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, wire, masonry comb, or steel plate that is embedded in a masonry section normal to the applied force, for the purpose of increasing usable compressive strain in the masonry.

Lateral Tie: Loop of reinforcing bar or wire enclosing longitudinal reinforcement in a column.

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.

Cavity Wall: Wall containing continuous air space of minimum width of 2 in. (51 mm) and of maximum width of 4-1/2 in. (114 mm) between wythes which are tied with metal ties.

Wall Frame: A moment resisting frame of masonry beams and masonry columns within a plane, with special reinforcement details and connections which provide resistance to lateral and gravity loads.

Wythe: A continuous vertical section of a wall, one masonry unit in thickness.

8.1.4 NOTATIONS:

A_b = tensile stress area of an anchor bolt, in.^{2} (mm^2).

A_n = net cross-sectional area of masonry, in.^{2} (mm^2).

A_p = projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations, in.^{2} (mm^2).

A_s = cross-sectional area of reinforcement, in.^{2} (mm^2).

A_{sh} = total cross-sectional area of rectangular tie reinforcement for the confined core of a wall frame section, in.^{2} (mm^2).

a = length of compressive stress block, in. (mm).

B_a = design axial strength of an anchor bolt, lb (N).

B_v = design shear strength of an anchor bolt, lb (N).

b_a = factored axial force on an anchor bolt, lb (N).

b_v = factored shear force on an anchor bolt, lb (N).

b_w = web width, in. (mm).

c = distance from the fiber of maximum compressive strain to the neutral axis, in. (mm).

d = effective depth of member, in. (mm).

d_b = diameter of reinforcement, in. (mm).

d_{bb} = diameter of the largest beam longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, in. (mm).

d_{bp} = diameter of the largest column (pier) longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, in. (mm).

E_m = modulus of elasticity of masonry, psi (MPa).

E_s = modulus of elasticity of reinforcement, psi (MPa).

E_v = modulus of rigidity of masonry, psi (MPa).

f'_g = specified compressive strength of grout, psi (MPa).

f'_m = specified compressive strength of masonry at the age of 28 days, unless a different age is specified, psi (MPa).

f_r = modulus of rupture of masonry, psi (MPa).

f_y = specified yield strength of the reinforcement or the anchor bolt as applicable, psi (MPa).

h = height of the member between points of lateral support, in. (mm).

h_b = beam depth in the plane of the wall frame, in. (mm).

h_c = cross-sectional dimension of grouted core of wall frame member measured center to center of confining reinforcement, in. (mm).

h_p = pier depth in the plane of the wall frame, in. (mm).

I_{cr} = moment of inertia of the cracked section, in.^{4} (mm^4).

I_{eff} = effective moment of inertia, in.^{4} (mm^4).

I_n = moment of inertia of the net cross-sectional area of a member, in.^{4} (mm^4).

K = the lesser of the masonry cover, clear spacing between adjacent reinforcement, or 3 times d_b, in. (mm).

L = length of coupling beam between coupled shear walls, in. (mm).

ℓ_b = effective embedment length of anchor bolt, in. (mm).

ℓ_{be} = anchor bolt edge distance, in. (mm).

ℓ_d = development length, in. (mm).

ℓ_{dh} = equivalent development length for a standard hook, in. (mm).

ℓ_{ld} = minimum lap splice length, in. (mm).

M = moment on a masonry section due to unfactored load, in.-lb (N-mm).

M_a = maximum moment in member due to the applied loading for which deflection is computed, in.-lb (N-mm).

M_{cr} = cracking moment strength of the masonry, in.-lb (N-mm).

M_d = design moment strength, in.-lb (N-mm).

M_u = required flexural strength due to factored loads, in.-lb (N-mm).

M_1, M_2 = nominal moment strength at the ends of the coupling beam, in.-lb (N-mm).

N_v = force acting normal to shear surface, lb (N).

P = axial force on a masonry section due to unfactored loads, lb (N).

P_n = nominal axial load strength, lb (N).

P_u = required axial strength due to factored loads, lb (N).

r = radius of gyration, in. (mm).

S = section modulus based on net cross-sectional area of a wall, in.^{3} (mm^3).

s = spacing of lateral reinforcement in wall frame members, in. (mm).

t = specified wall thickness dimension or least lateral dimension of a column, in. (mm).

V = shear on a masonry section due to unfactored loads, lb (N).

V_m = shear strength provided by masonry, lb (N).

V_n = nominal shear strength, lb (N).

V_s = shear strength provided by shear reinforcement, lb (N).

V_u = required shear strength due to factored loads, lb (N).

ρ = ratio of the area of reinforcement to the net cross-sectional area of masonry in 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 net cross-sectional area of masonry in a plane perpendicular to the reinforcement.

ϵ_{mu} = maximum usable compressive strain of masonry, in./in. (mm/mm).

ϕ = strength reduction factor.

8.2 CONSTRUCTION REQUIREMENTS:

8.2.1 GENERAL: Masonry shall be constructed in accordance with the requirements of Ref. 8-2. Materials shall conform to the requirements of one of the standards listed in Sec. 8.1.2.1.

8.2.2 QUALITY ASSURANCE: Inspection and testing of masonry materials and construction shall comply with the requirements of Sec. 1.6.

8.3 GENERAL DESIGN REQUIREMENTS:

8.3.1 SCOPE: Masonry structures and components of masonry structures shall be designed in accordance with the requirements of reinforced masonry design, plain (unreinforced) masonry design, or empirical design, subject to the limitations of this section.

8.3.2 EMPIRICAL MASONRY DESIGN: The requirements of Chapter 9 of Ref. 8-1 shall apply to the empirical design of masonry.

8.3.3 PLAIN (UNREINFORCED) MASONRY DESIGN:

8.3.3.1: In the design of plain (unreinforced) masonry members, the flexural tensile strength of masonry units, mortar and grout in resisting design loads shall be permitted.

8.3.3.2: In the design of plain masonry members, stresses in reinforcement shall not be considered effective in resisting design loads.

8.3.3.3: Plain masonry members shall be designed to remain uncracked.

8.3.4 REINFORCED MASONRY DESIGN: In the design of reinforced masonry members, stresses in reinforcement shall be considered effective in resisting design loads.

8.3.5 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A shall comply with the requirements of Sec. 8.3.2 (empirical masonry design), 8.3.3 (plain masonry design), or 8.3.4 (reinforced masonry design).

8.3.6 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. 8.3.3 or Sec. 8.3.4.

8.3.7 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.

8.3.7.1: 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 (13 mm) lateral ties shall be provided in the top 5 in. (127 mm) of the column.

8.3.7.2: Vertical reinforcement of at least 0.20 in.^{2} (129 mm^2) 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 0.20 in.^{2} (129 mm^2) in cross section shall be provided as follows:

a. At the bottom and top of wall openings extending not less than 24 in. (610 mm) nor 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 120 in. (3048 mm) 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.

8.3.7.3: Where stack bond is used, the minimum horizontal reinforcement shall be 0.0007 times the gross cross-sectional area of the wall. This requirement shall be satisfied with uniformly distributed joint reinforcement or with horizontal reinforcement spaced not over 48 in. (1219 mm) and fully embedded in grout or mortar.

8.3.7.4 Multiple Wythe Walls Not Acting Compositely: At least one wythe of a cavity wall shall be reinforced masonry designed in accordance with Sec. 8.3.4. The other wythe shall be reinforced with a minimum of one W1.7 wire per 4-in. (102 mm) nominal wythe thickness and spaced at intervals not exceeding 16 in. (406 mm). The wythes shall be tied in accordance with Ref. 8-1, Sec. 5.8.2.2.

8.3.7.5 Walls Separated from the Basic Structural System: Masonry walls, laterally supported perpendicular to their own plane but otherwise structurally isolated on three sides from the basic structural system, shall have minimum horizontal reinforcement of 0.007 times the gross cross-sectional area of the wall. This requirement shall be satisfied with uniformly distributed joint reinforcement or with horizontal reinforcement spaced not over 48 in. (1200 mm) and fully embedded in grout or mortar.

8.3.7.6 Material Requirements: Structural clay load-bearing wall tile (Ref. 8-11) shall not be used.

8.3.8 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.

8.3.8.1: Neither Type N mortar nor masonry cement shall be used as part of the basic structural system.

8.3.8.2: 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 48 in. (1219 mm). Except for joint reinforcement, the diameter of reinforcement shall not be less than 3/8 in. (10 mm). Reinforcement shall be continuous around wall corners and through intersections, unless the intersecting walls are separated. Only horizontal reinforcement that is continuous in the wall or element shall be included in computing the area of horizontal reinforcement. Reinforcement spliced in accordance with Sec. 8.4.5.6 shall be considered as continuous reinforcement.

8.3.8.3: For columns subjected to axial forces due to earthquake loads on the building, ties shall be spaced not more than 8 in. (203 mm) on center for the full height of the column. All other columns shall have ties spaced a maximum of 8 in. (203 mm) on center over regions at their tops and bottoms. The dimension of these regions along the length of the columns shall be the larger of one-sixth of the column height, 18 in. (457 mm), or the maximum column cross-sectional dimension. Lateral tie spacing for the remainder of these columns shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, the lesser column cross-sectional dimension, or 18 in. (457 mm). Lateral ties for compression reinforcement shall be embedded in grout and shall be No. 3 (10 mm) or larger.

8.3.8.4: Where masonry is laid in stack 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 in. (610 mm).

8.3.8.5 Minimum Dimensions:

8.3.8.5.1: The nominal thickness of masonry bearing walls shall not be less than 6 in. (152 mm). Nominal 4-in. (102 mm) thick load-bearing reinforced hollow clay unit masonry walls with a maximum unsupported height or length to thickness ratio of 27 are permitted to be used provided the net area unit strength exceeds 8,000 psi (55 MPa), units are laid in running bond, bar sizes do not exceed 1/2 in. (13 mm) with not more than two bars or one splice in a cell and joints are not raked.

8.3.8.5.2: The nominal dimensions of a masonry column shall not be less than 12 in. (305 mm).

8.3.8.6 Shear Walls:

8.3.8.6.1: Reinforcement required to resist in-plane shear shall be placed horizontally, shall be uniformly distributed, and shall be embedded in mortar or grout. The maximum spacing of reinforcement in each direction shall not exceed one-third the length of the wall, one-third the height of the wall, or 48 in. (1219 mm).

8.3.8.6.2: Reinforcement required to resist in-plane shear shall be terminated with a standard 180-degree hook or developed by embedment beyond the end of the wall. Grout placement shall not be obstructed. Wall reinforcement terminating in columns or beams shall be developed in these elements.

8.3.8.7: Where concrete abuts structural masonry and the joint between the materials is not designed as a separation joint, the concrete shall be roughened so that the average height of aggregate exposure is 1/8-in. (3 mm) and shall be bonded to the masonry in accordance with these provisions as if it were masonry. Vertical joints not intended to act as separation joints shall be crossed by horizontal reinforcement as required by Sec. 8.3.8.2.

8.3.9 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.

8.3.9.1 Construction Requirements: Construction procedures or admixtures shall be used to minimize cracking of grout and to maximize bond.

8.3.9.2 Reinforced Hollow Unit Masonry: 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 to prevent displacement during grouting. Horizontal wall reinforcement shall be held in position to prevent displacement during grouting.

8.3.9.3 Stack Bond Construction: Masonry laid in stack bond shall conform to the following requirements:

8.3.9.3.1: For masonry that is not part of the basic structural system, the minimum ratio of horizontal reinforcement shall be 0.0015 and the maximum spacing of horizontal reinforcement shall be 24 in. (610 mm). For masonry that is part of the basic structural system, the minimum ratio of horizontal reinforcement shall be 0.0025 and the maximum spacing of horizontal reinforcement shall be 16 in. (406 mm). For the purpose of calculating this ratio, joint reinforcement shall not be considered.

8.3.9.3.2: Reinforced hollow unit construction shall be grouted solid and all head joints shall be made solid by the use of open end units.

8.3.10 PROPERTIES OF MATERIALS:

8.3.10.1: Unless otherwise determined by test, steel reinforcement modulus of elasticity (E_s) shall be taken to be 29,000,000 psi (200,000 MPa).

8.3.10.2: The modulus of elasticity of masonry (E_m) shall be determined in accordance with Eq. 8.3.10.2 or shall be based on the modulus of elasticity determined by prism test and taken between 0.05 and 0.33 times the masonry prism strength.

$$E_m = 750 f_m' \qquad (8.3.10.2)$$

where:

E_m = modulus of elasticity of masonry, psi, and

f_m' = specified compressive strength of masonry, psi.

The metric equivalent of Eq. 8.3.10.2 is the except that E_m and f_m' are in MPa.

8.3.10.3: The modulus of rigidity of masonry, E_v, shall be taken equal to 0.4 times the modulus of elasticity of masonry, E_m.

8.3.10.4 Masonry Compressive Strength:

8.3.10.4.1: The specified compressive strength of masonry, f_m', shall equal or exceed 1,500 psi (10 MPa).

8.3.10.4.2: The value of f_m' used to determine nominal strength values in this chapter shall not exceed 4,000 psi (28 MPa) for concrete masonry and shall not exceed 6,000 psi (41 MPa) for clay masonry.

8.3.10.5 Modulus of Rupture:

8.3.10.5.1 Running Bond Masonry: The modulus of rupture, f_r, for masonry shall be taken from Table 8.3.10.5.1.

TABLE 8.3.10.5.1
Modulus of Rupture for Masonry Laid in Running Bond (f_r)

Masonry type	Mortar types, psi (MPa)			
	Portland cement/lime		Masonry cement and air-entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	80 (0.55)	60 (0.41)	48 (0.33)	30 (0.21)
Hollow units[a]				
Ungrouted	50 (0.34)	38 (0.26)	30 (0.21)	18 (0.12)
Fully grouted	136 (0.94)	116 (0.80)	82 (0.57)	52 (0.36)
Parallel to bed joints in running bond				
Solid units	160 (1.10)	120 (0.83)	96 (0.66)	60 (0.41)
Hollow units				
Ungrouted and partially grouted	100 (0.69)	76 (0.52)	60 (0.41)	38 (0.26)
Fully grouted (running bond masonry)	160 (1.10)	120 (0.83)	96 (0.66)	60 (0.41)

[a] For partially grouted masonry, modulus of rupture values shall be determined on the basis of linear interpolation between hollow units which are fully grouted and hollow units which are ungrouted based on amount (percentage) of grouting.

8.3.10.5.2 Stack Bond Masonry: For grouted stack bond masonry, the modulus of rupture, f_r, parallel to the bed joints shall be taken as 250 psi (1.7 MPa) and shall be based only on the continuous grout cross section. For ungrouted stack bond masonry, the modulus rupture, f_r, normal to head joints shall be taken as zero.

8.3.10.6 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 (400 MPa).

8.3.11 SECTION PROPERTIES:

8.3.11.1: Member strength shall be computed using section properties based on the net cross-sectional area of the member under consideration.

8.3.11.2: Section properties shall be based on specified dimensions.

8.3.12 PLATE, HEADED AND BENT BAR ANCHOR BOLTS:

8.3.12.1: The design axial strength, B_a, for plate anchors, headed anchor bolts and bent bar anchor bolts (J or L) embedded in masonry shall be the lesser of Eq. 8.3.12.1-1 (strength governed by masonry) or Eq. 8.3.12.1-2 (strength governed by steel):

$$b_a \leq \phi 4 A_p \sqrt{f_m'} \qquad (8.3.12.1\text{-}1)$$

$$B_a \leq \phi 0.1 A_b f_y \qquad (8.2.12.1\text{-}2)$$

where:

B_a = design axial strength of an anchor bolt, lb;

ϕ = strength reduction factor where $\phi = 0.5$ for Eq. 8.3.12.1-1 and $\phi = 0.9$ for Eq. 8.3.12.1-2;

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};

f_m' = compressive strength of the masonry, psi; and

f_y = specified yield strength of the reinforcement or the anchor bolt as applicable, psi.

The metric equivalent of Eq. 8.3.12.1-1 is:

$$B_a \leq \phi\left(0.33 A_p \sqrt{f'_m}\right)$$

where B_a is in N, A_p is in mm^2, and f'_m is in MPa. The metric equivalent of Eq. 8.3.12.1-2 is the same except that B_a is in N, A_b is in mm^2, and f_y is in MPa.

8.3.12.1.1: The area A_p in Sec. 8.1.4 shall be the lesser of Eq. 8.3.12.1.1-1 or Eq. 8.3.12.1.1-2:

$$A_p = \pi \ell_b^2 \qquad (8.3.12.1.1\text{-}1)$$

$$A_p = \pi \ell_{be}^2 \qquad (8.3.12.1.1\text{-}2)$$

where:

A_p = projected area on the masonry surface of a right circular cone, in.^{2};

ℓ_b = effective embedment length of the anchor bolt, in.; and

ℓ_{be} = anchor bolt edge distance, in..

The metric equivalents of Eq. 8.3.12.1.1-1 and Eq. 8.3.12.1.1-2 are the same except that A_p is in mm^2 and ℓ_b and ℓ_{be} are in mm.

Where the areas 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. 8.3.12.1.1-1 or Eq. 8.3.12.1.1-2, whichever is less.

8.3.12.1.2: 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.

8.3.12.1.3: 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.

8.3.12.1.4: The minimum effective embedment length shall be 4 bolt diameters or 2 in. (51 mm), whichever is greater.

8.3.12.2: Where the anchor bolt edge distance, ℓ_{be}, equals or exceeds 12 bolt diameters, the design shear strength, (B_v), shall be the lesser of the values given by Eq. 8.3.12.2-1 (strength governed by masonry) or Eq. 8.3.12.2-2 (strength governed by steel):

$$B_v = \phi 1750 \sqrt[4]{f_m' A_b} \qquad (8.3.12.2\text{-}1)$$

$$B_v = \phi 0.6 A_b f_y \qquad (8.3.12.2\text{-}2)$$

where:

ϕ = strength reduction factor where $\phi = 0.5$ for Eq. 8.3.12.2-1 and $\phi = 0.9$ for Eq. 8.3.12.2-2;

A_b = cross-sectional area of the anchor bolt, in.^{2}; and

f_m' = specified compressive strength of the masonry, psi.

f_y = specified yield strength of the reinforcement or the anchor bolt as applicable, psi.

The metric equivalent of Eq. 8.3.12.2-1 is:

$$B_v = \phi 5350 \sqrt[4]{f_m' A_b}$$

where A_b is in mm^2 and f_m' and f_y are in MPa. The metric equivalent of Eq. 8.3.12.2-2 is the same as that above except that A_b is in mm^2 and f_y is in MPa.

Where the anchor bolt edge distance, ℓ_{be}, is less than 12 bolt diameters, the value of B_v in Eq. 8.3.12.2-1 shall be reduced by linear interpolation to zero at an ℓ_{be} distance of 1 in. (25 mm).

8.3.12.3: Anchors subjected to combined shear and tension shall be designed to satisfy Eq. 8.3.12.3:

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} \le 1 \qquad (8.3.12.3)$$

where:

b_a = design axial force on the anchor, lb (N);

B_a = design axial strength of the anchor, lb (N);

b_v = design shear force on the anchor, lb (N); and

B_v = design shear strength of the anchor, lb (N).

8.4 DETAILS OF REINFORCEMENT:

8.4.1 GENERAL:

8.4.1.1: Details of reinforcement shall be shown or covered by notes on the contract documents.

8.4.1.2: Reinforcing bars shall be embedded in grout.

8.4.2 SIZE OF REINFORCEMENT:

8.4.2.1: The maximum size of reinforcing bars used in masonry shall be No. 9 (29 mm) and the bar size number shall not exceed the nominal wall thickness in inches (mm). The maximum area of a reinforcing bars placed in hollow unit construction shall be 4 percent of the cell area.

8.4.2.2: The diameter of reinforcing bars shall not exceed one-quarter of the least clear dimension of the cell, bond beam or collar joint in which it is placed.

8.4.2.3: Joint reinforcement longitudinal and cross wires shall be minimum W1.1, (0.011 mm^2) and shall not exceed one-half the joint thickness.

8.4.3 PLACEMENT LIMITS FOR REINFORCEMENT:

8.4.3.1: The clear distance between parallel reinforcing bars shall not be less than the nominal diameter of the bars nor less than 1 in. (25 mm).

8.4.3.2: 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 in. (38 mm).

8.4.3.3: The clear distance limitations between reinforcing bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.4.3.4: Reinforcing bars shall not be bundled.

8.4.3.5: Reinforcement embedded in grout shall have a thickness of grout between the reinforcement and masonry units not less than 1/2 in. (13 mm) for fine grout or 3/4 in. (19 mm) for coarse grout.

8.4.4 COVER FOR REINFORCEMENT:

8.4.4.1: Reinforcing bars shall have a minimum masonry cover not less than 2-1/2 d_b nor less than the following:

a. Where the masonry face is exposed to earth or weather, 2 in. (51 mm) for bars larger than No. 5 (16 mm) and 1-1/2 in. (38 mm) for No. 5 (16 mm) bar or smaller.

b. Where the masonry is not exposed to earth or weather, 1-1/2 in. (38 mm).

8.4.4.2: Longitudinal wires of joint reinforcement shall be fully embedded in mortar or grout with a minimum cover of 1/2 in. (13 mm) when exposed to earth or weather and 3/8 in. (10 mm) 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. 8.4.1.1.)

8.4.4.3: Wall ties, anchors and inserts, except anchor bolts not exposed to the weather or moisture, shall be protected from corrosion.

8.4.5 DEVELOPMENT OF REINFORCEMENT:

8.4.5.1 General: The calculated tension or compression in the reinforcement where masonry reinforcement 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.

8.4.5.2 Embedment of Reinforcing Bars and Wires in Tension: The embedment length, ℓ_d, of reinforcing bars and wire shall be determined by Eq. 8.4.5.2 but shall not be less than 12 in. (305 mm) for bars and 6 in. (152 mm) for wire:

$$l_d = \left(\frac{1}{\phi}\right)\left(\frac{0.15 d_b^2 f_y}{K\sqrt{f_m'}}\right) \leq \frac{52 d_b}{\phi} \qquad (8.4.5.2)$$

where:

ϕ = 0.8, strength reduction factor;

d_b = diameter of the reinforcement, in.;

K = the lesser of the masonry cover, clear spacing between adjacent reinforcement, or 3 times d_b, in.;

f_m' = specified compressive strength of masonry, psi; and

f_y = specified yield strength of the reinforcement, psi.

The metric equivalent of Eq. 8.4.5.2 is:

$$l_d = \left(\frac{1}{\phi}\right)\left(\frac{1.8 d_b^2 f_y}{K\sqrt{f_m'}}\right) \leq \frac{52 d_b}{\phi}$$

where ℓ_d and d_b are in mm and f_y and f_m' are in MPa.

8.4.5.3 Standard Hooks:

8.4.5.3.1: The term standard hook as used in this code shall mean one of the following:

8.4.5.3.1.1: A 180-degree turn plus extension of at least 4 bar diameters but not less than 2-1/2 in. (64 mm) at free end of bar.

8.4.5.3.1.2: A 135-degree turn plus extension of at least 6 bar diameters at free end of bar.

8.4.5.3.1.3: 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.

8.4.5.4 Minimum Bend Diameter for Reinforcing Bars:

8.4.5.4.1: 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 8.4.5.4.1.

TABLE 8.4.5.4.1
Minimum Diameters of Bend

Bar Size	Grade	Minimum Bend
No. 3 (10 mm) through No. 7 (22 mm)	40	5 bar diameters
No. 3 (10 mm) through No. 8 (25 mm)	50 or 60	6 bar diameters
No. 9 (29 mm)	50 or 60	8 bar diameters

8.4.5.4.2: The equivalent embedment length for standard hooks in tension, ℓ_{dh}, shall be as follows:

$$\ell_{dh} = 13d_b \qquad (8.4.5.4.2)$$

where d_b = diameter of the reinforcement, in. The metric equivalent of Eq. 8.4.5.4.2 is the same except that d_b is in mm.

8.4.5.4.3: The effect of hooks for bars in compression shall be neglected in design computations.

8.4.5.5 Development of Shear Reinforcement:

8.4.5.5.1 Bar and Wire Reinforcement:

8.4.5.5.1.1: 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.

8.4.5.5.1.2: 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 (16 mm) 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 ends, each bend in the continuous portion of a transverse U-stirrup shall enclose a longitudinal bar.

8.4.5.5.2 Wire Fabric:

8.4.5.5.2.1: For each leg of welded wire fabric forming simple U-stirrups, there shall be either:

a. Two longitudinal wires located at a 2 in. (51 mm) 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 in. (51 mm) 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 $8d_b$.

8.4.5.5.2.2: For each end of a single leg stirrup of welded smooth or deformed wire fabric, there shall be two longitudinal wires spaced at a minimum of 2 in. (51 mm) with the inner wire placed at a distance at least *d*/4 or 2 in. (51 mm) 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.

8.4.5.6 Splices of Reinforcement: Lap splices, welded splices, or mechanical connections shall be in accordance with the provisions of this section. All welding shall conform to Ref. 8-36.

8.4.5.6.1 Lap Splices: Lap splices shall not be used in plastic hinge zones. The length of the plastic hinge zone shall be taken as at least $0.15h$.

8.4.5.6.1.1: The minimum length of lap, ℓ_{ld}, for bars in tension or compression shall be equal to the development length, ℓ_d, as determined by Eq. 8.4.5.2 but shall not be less than 12 in. (305 mm) for bars and 6 in (152 mm) for wire.

8.4.5.6.1.2: 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 in. (203 mm).

8.4.5.6.1.3: The minimum length of lap required in Eq. 8.4.5.6.1.1 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.

8.4.5.6.2 Welded Splices: A welded splice shall be capable of developing in tension 125 percent of the specified yield strength, f_y, of the bar.

8.4.5.6.3 Mechanical Connections: 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.

8.4.5.6.4 End Bearing Splices:

8.4.5.6.4.1: 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.

8.4.5.6.4.2: 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.

8.4.5.6.4.3: End bearing splices shall be used only in members containing closed ties, closed stirrups or spirals.

8.5 STRENGTH AND DEFORMATION REQUIREMENTS:

8.5.1 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.

8.5.2 REQUIRED STRENGTH: The required strength shall be determined in accordance with Chapters 2 and 3.

8.5.3 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 8.5.3.

TABLE 8.5.3
Strength Reduction Factor ϕ

Flexure without axial load	Reinforced masonry Plain masonry	$\phi = 0.85$ $\phi = 0.60$
Axial load and axial load with flexure	Reinforced masonry[a] Reinforced masonry wall frames Plain masonry	$\phi = 0.65$ $\phi = 0.85 - (P_u/A_n f_m')$, but not less than 0.65 $\phi = 0.60$
Shear	Reinforced masonry	$\phi = 0.80$
Shear	Plain masonry	$\phi = 0.80$
Reinforcement development length and splices		$\phi = 0.80$
Anchor bolt strength as governed by steel		$\phi = 0.90$
Anchor bolt strength as governed by masonry		$\phi = 0.50$
Bearing		$\phi = 0.60$

[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.

8.5.4 DEFORMATION REQUIREMENTS:

8.5.4.1: Masonry structures shall be designed so the design story drift, Δ, does not exceed the allowable story drift, Δ_a, obtained from Table 2.2.7.

8.5.4.1.1: Cantilever shear walls shall be proportioned such that the maximum displacement, δ_{max}, at Level n does not exceed $0.01h_n$.

8.5.4.2: Deflection calculations for plain masonry members shall be based on uncracked section properties.

8.5.4.3: Deflection calculations for reinforced masonry members shall be based on an effective moment of inertia in accordance with the following:

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1-\left(\frac{M_{cr}}{M_a}\right)^3\right] \leq I_n \qquad (8.5.4.3)$$

where:

$M_{cr} =$ Sf_r;

$M_{cr} =$ 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, in.^{3}; and

f_r = modulus of rupture of masonry, psi.

The metric equivalent of Eq. 8.5.4.3 is the same except that M_{cr} and M_a are in (N-mm), I_{cr} and I_n are in mm^4, S is in mm^3, and f_r is in MPa.

8.5.4.4: The calculated deflection shall be multiplied by C_d for determining drift.

8.6 FLEXURE AND AXIAL LOADS:

8.6.1 SCOPE: This section shall apply to the design of masonry members subject to flexure or axial loads or to combined flexure and axial loads.

8.6.2 DESIGN REQUIREMENTS OF REINFORCED MASONRY MEMBERS:

8.6.2.1: 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 block. Masonry stress of 0.85 times the specified compressive strength, f_m', shall be assumed to be uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance a = 0.85 c from the fiber of maximum compressive strain.

8.6.2.2: The ratio of reinforcement, ρ, shall not exceed the ratio that will cause the following critical strain conditions.

The critical strain condition exists at a cross section when the strain in the extreme tension reinforcement is 5 times the strain corresponding to its yield strength, f_y, and the strain in the extreme compression fiber is 0.002.

The reinforcement ratio, ρ, shall be calculated using unfactored axial loads. For calculation, the stress in the tension reinforcement shall be assumed to be 1.25 f_y. Tension in the masonry shall be neglected. The compressive strength in the masonry shall be assumed to vary linearly from zero at the neutral axis to f_m' at the extreme compression fiber. Compressive steel stress shall be based on a linear strain distribution.

8.6.2.3: 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.

8.6.3 DESIGN OF PLAIN (UNREINFORCED) MASONRY MEMBERS:

8.6.3.1: 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.

8.6.3.2: Strain in masonry shall be assumed directly proportional to the distance from the neutral axis.

8.6.3.3: Flexural tension in masonry shall be assumed directly proportional to strain.

8.6.3.4: Flexural compressive stress in combination with axial compressive stress in masonry shall be assumed directly proportional to strain. Maximum compressive stress shall not exceed $0.85f_m'$.

8.6.3.5: Design axial load strength shall be in accordance with Eq. 8.6.3.5-1 or Eq. 8.6.3.5-2.

$$\phi P_n = \phi A_n f_m' \left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } h/r < 99 \qquad (8.6.3.5\text{-}1)$$

$$\phi P_n = \phi A_n f_m' \left(\frac{70r}{h}\right)^2 \text{ for } h/r \geq 99 \qquad (8.6.3.5\text{-}2)$$

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, inches.

The metric equivalents for Eq. 8.6.3.5-1 and Eq. 8.6.3.5-2 are the same except that A_n is in mm^2, f'_m is in MPa, and h and r are in mm.

8.7 SHEAR:

8.7.1 SCOPE: Provisions of this section shall apply for design of members subject to shear.

8.7.2 SHEAR STRENGTH:

8.7.2.1: Design of cross sections subjected to shear shall be based on:

$$V_u \leq \phi V_n \qquad (8.7.2.1)$$

where:

V_u = factored shear force at the section considered, lb;

ϕ = 0.80, strength reduction factor; and

V_n = nominal shear strength, lb.

The metric equivalent of Eq. 8.7.2.1 is the same except that V_u and V_n are in N.

8.7.2.2: The nominal shear strength shall exceed the shear corresponding to the development of the nominal flexural strength of the member except that the nominal shear strength need not exceed 2.5 times V_u.

8.7.3 DESIGN OF REINFORCED MASONRY MEMBERS:

8.7.3.1: Nominal shear strength, V_n, shall be computed as follows:

$$V_n = V_m + V_s \qquad (8.7.3.1\text{-}1)$$

where:

V_n = nominal shear strength, lb;

V_m = nominal shear strength provided by masonry, lb; and

V_s = shear strength provided by reinforcement, lb.

The metric equivalent for Eq. 8.7.3.1-1 is the same except that V_n, V_m, and V_s are in N.
For $M/Vd < 0.25$:

$$V_{n(\max)} = 6\sqrt{f'_m}\,A_n \qquad (8.7.3.1\text{-}2)$$

For $M/Vd \geq 1.00$:

$$V_{n_{(\max)}} = 4\sqrt{f'_m}\,A_n \qquad (8.7.3.1\text{-}3)$$

where:

$V_{n(max)}$ = maximum nominal shear strength, lb;

A_n = net cross-sectional area of the masonry, in.^{2};

f'_m = specified compressive strength of the masonry, psi;

M = moment on the masonry section due to unfactored design loads, in.-lb;

V = shear on the masonry section due to unfactored loads, lb; and

d = depth of member, inches.

Values of M/Vd between 0.25 and 1.0 may be interpolated.
The metric equivalent of Eq. 8.7.3.1-2 is:

$$V_{n(max)} = 0.5\sqrt{f'_m}\,A_n$$

and the metric equivalent of Eq. 8.7.3.1-3 is:

$$V_{n(max)} = 0.33\sqrt{f'_m}\,A_n$$

where $V_{n(max)}$ is in N, A_n is in mm^2, f'_m is in MPa, M is in N-mm, and d is in mm.

8.7.3.2: Shear strength, V_m, provided by masonry for sections not in a plastic hinge zone shall be as follows:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P \qquad (8.7.3.2\text{-}1)$$

where M/Vd need not be taken greater than 1.0 and

V_m	=	shear strength provided by masonry, lb;
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.

Shear strength provided by masonry for sections in a plastic hinge zone shall be as follows:

$$V_m = 1.0\, A_n \sqrt{f_m'} + 0.3P \qquad (8.7.3.2\text{-}2)$$

The metric equivalent of Eq. 8.7.3.2-1 is:

$$V_m = 0.083\left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f_m'} + 0.25P$$

and the metric equivalent of Eq. 8.7.3.2.2 is:

$$V_m = 0.083 A_n \sqrt{f_m'} + 0.3P$$

where V_m and P are in N, M is in N-mm, f'_m is in MPa, d is in mm, and A_n is in mm^2.

8.7.3.3: Nominal shear strength, V_s, provided by reinforcement shall be as follows:

$$V_s = 0.5 \rho_h f_y A_n \qquad (8.7.3.3)$$

where:

ρ_h	=	ratio of the area of shear reinforcement;
f_y	=	specified yield strength of the reinforcement or the anchor bolt as applicable, psi; and
A_n	=	net cross-sectional area of masonry, in.^{2}.

The metric equivalent of Eq. 8.7.3.3 is the same except that f_y is in MPa and A_n is in mm^2.

8.7.4 DESIGN OF PLAIN (UNREINFORCED) MASONRY MEMBERS:

8.7.4.1: Nominal shear strength, V_n, shall be the lesser of the following:

a. $1.5\sqrt{f_m'}A_n$, lb (the metric equivalent is $0.125\sqrt{f_m'}A_n$, N, where f_m' is in MPa and A_n is in mm^2);

b. $120A_n$, lb (the metric equivalent is $0.83A_n$, N, where A_n is in mm^2);

c. $37\ A_n + 0.3\ N_v$ for running bond masonry not grouted solid, lb (the metric equivalent is $0.26A_n + 0.3N_v$ when A_n is in mm^2 and N_v is in N);

   $37\ A_n + 0.3\ N_v$ for stack bond masonry with open end units and grouted solid, lb (the metric equivalent is $0.26A_n + 0.3N_v$ when A_n is in mm^2 and N_v is in N);

   $60\ A_n + 0.3\ N_v$ for running bond masonry grouted solid, lb (the metric equivalent is $0.414A_n + 0.3N_v$ when A_n is in mm^2 and N_v is in N); and

   $15\ A_n$ for stack bond other than open end units grouted solid, lb (the metric equivalent is $0.103A_n + 0.3N_v$ when A_n is in mm^2 and N_v is in N)

   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.

8.8 SPECIAL REQUIREMENTS FOR BEAMS:

8.8.1: The spacing between lateral supports shall be determined by the requirements for out of-plane loading, but it shall not exceed the following:

a. 50 times the least width of beams with flexural reinforcement of a minimum of two bars with confinement reinforcement.

b. 32 times the least width of beams which do not comply with the reinforcement requirements of Sec. 8.8.1.a.

8.8.2: The effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

8.8.3: At any section of a flexural member where positive reinforcement is required by analysis, the ratio ρ shall not be less than $120/f_y$ (the metric equivalent is $0.83/f_y$ where f_y is in MPa) except that this minimum positive steel reinforcement ratio need not be satisfied if the area of positive and negative reinforcement provided at every section is one third greater than that required by analysis and the Seismic Performance Category is A, B, or C.

Where a concrete floor provides a flange and where the beam web is in tension, the ratio ρ shall be computed using the web width.

8.8.4 DEEP FLEXURAL MEMBERS:

8.8.4.1: 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.

8.8.4.2: Minimum flexural tension reinforcement shall conform to Sec. 8.8.3.

8.8.4.3: Uniformly distributed horizontal and vertical reinforcement shall be provided throughout the length and depth of deep flexural members such that the reinforcement ratios in both directions are at least 0.001. Distributed flexural reinforcement is to be included in the determination of the actual reinforcement ratios.

8.9 SPECIAL REQUIREMENTS FOR COLUMNS:

8.9.1: 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.

8.9.2: There shall be a minimum of four longitudinal bars in columns.

8.9.3: Lateral ties shall be provided to resist shear and shall comply with the following:

a. Lateral ties shall be at least 1/4 in. (6 mm) 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 in. (152 mm) 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. Minimum 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 in. (76 mm) below lowest reinforcement in shallowest of such beams or brackets.

8.10 SPECIAL REQUIREMENTS FOR WALLS:

8.10.1: The nominal flexural strength of the wall for out-of-plane flexure shall be at least equal to 1.3 times the cracking moment strength of the wall.

8.11 SPECIAL REQUIREMENTS FOR SHEAR WALLS:

8.11.1 REINFORCEMENT: Spacing of horizontal reinforcement shall not exceed six times nominal wall thickness or 48 in. (1220 mm), whichever is less.

8.11.2 CONFINEMENT OF COMPRESSIVE STRESS ZONE:

8.11.2.1: Confinement ties, when used in the design procedure, shall be provided in the plastic hinge zone of shear walls.

8.11.2.2: The minimum length of the confinement region, measured along the length of the wall, shall be three times the thickness of the wall, pier, or column.
The confined compressive zone shall have a longitudinal dimension of the lesser of one-quarter the distance from the point of maximum moment to the point of zero moment or one-sixth of the length of the member.

8.11.2.3: Confinement ties shall comply with the requirements of Sec. 8.9.3, shall have a closed perimeter terminated by a standard 135-degree or 180-degree hook and shall consist of a minimum of No. 3 (10 mm) bars at a maximum 8-in. (203 mm) vertical spacing or equivalent.

8.11.3 FLANGED SHEAR WALLS:

8.11.3.1: Wall intersections shall be considered effective in transferring shear when the following requirements are met:

a. The face shells of hollow masonry units shall be removed and the intersection shall be fully grouted.

b. Solid units shall be laid in running bond and 50 percent of the masonry units at the intersection shall be interlocked.

c. All horizontal reinforcement shall be continuous through the intersection.

8.11.3.2: Flange shall be considered effective in resisting applied loads.

8.11.3.3: The width of flange considered effective in compression on each side of the web shall be taken equal to 9 times the thickness of the web or shall be equal to the actual flange on either side of the web wall, whichever is less.

8.11.3.4: The width of flange considered effective in tension on each side of the web shall be taken equal to 3/4 of the wall height or shall be equal to the actual flange on either side of the web wall, whichever is less.

8.11.4 COUPLED SHEAR WALLS:

8.11.4.1 Design of Coupled Shear Walls: Structural members which provide coupling between shear walls shall be designed to reach their moment or shear nominal strength before either shear wall reaches its moment or shear nominal strength. Analysis of coupled shear walls shall conform to accepted principles of mechanics.

8.11.4.2 Shear Strength of Coupling Beams: The nominal shear strength, V_n, of the coupling beams shall exceed the shear calculated:

$$V_n \geq \frac{M_1 + M_2}{L} \tag{8.11.4.2}$$

where:

V_n	=	nominal shear strength, lb (N);
M_1 and M_2	=	nominal moment strength at the ends of the beam, lb-in. (N-mm); and
L	=	length of the beam between the shear walls, in. (mm).

The metric equivalent of Eq. 8.11.4.2 is the same except that V_n is in N, M_1,M_2 are in N-mm, and L is in mm.

The calculation of the nominal flexural moment shall include the reinforcement in reinforced concrete roof and floor system. The width of the reinforced concrete used for calculations of reinforcement shall be six times the floor or roof slab thickness.

8.12 WALL FRAMES:

8.12.1: 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.

8.12.2: 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 estimated stiffnesses of the members are reduced to their effective stiffnesses.

The stiffness used for the wall frame shall be that used for calculation of the effective period for calculation of lateral loading.

8.12.3: Flexural yielding of members shall be limited to the flexural beams at the face of the columns and to the bottom of the columns at the base of the building.

8.12.4 REINFORCEMENT:

8.12.4.1: 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.

8.12.4.2: Lap splices are permitted only within the center half of the member length.

8.12.4.3: 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 in. (610 mm) along the longitudinal axis.

8.12.4.4: Reinforcement shall have a specified yield strength of 60,000 psi (414 MPa). The actual yield strength shall not exceed 1.5 times the specified yield strength.

8.12.5 WALL FRAME BEAMS:

8.12.5.1: 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'.

8.12.5.2: Beams interconnecting vertical elements of the lateral load resisting system shall be limited to a reinforcement ratio of $0.15f_m'/f_y$ or 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.

8.12.5.3: Clear span for the beam shall not be less than 4 times its depth.

8.12.5.4: Nominal depth of the beam shall not be less than 4 units or 32 in. (813 mm), whichever is greater. The nominal depth to nominal width ratio shall not exceed 4.

8.12.5.5: Nominal width of the beams shall equal or exceed all of the following criteria:

a. 8 in. (203 mm),

b. width required by Sec. 8.8.1, and

c. 1/26 of the clear span between column faces.

8.12.5.6 Longitudinal Reinforcement:

8.12.5.6.1: Longitudinal reinforcement shall not be spaced more than 8 in. (203 mm) on center.

8.12.5.6.2: Longitudinal reinforcement shall be uniformly distributed along the depth of the beam.

8.12.5.6.3: Minimum reinforcement ratio shall be $130/f_y$ (the metric equivalent is $0.90/f_y$ where f_y is in MPa).

8.12.5.7 Transverse Reinforcement:

8.12.5.7.1: Transverse reinforcement shall be hooked around top and bottom longitudinal bars and shall be terminated with a standard 180-degree hook.

8.12.5.7.2: Within an end region extending one beam depth from wall 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.

8.12.5.7.3: The maximum spacing of transverse reinforcement shall not exceed 1/2 the nominal depth of the beam or that required for shear strength.

8.12.5.7.4: Minimum transverse reinforcement ratio shall be 0.0015.

8.12.6 WALL FRAME COLUMNS:

8.12.6.1: Factored axial compression force on the wall frame column shall not exceed 0.15 times the net cross-sectional area of the column, A_n, times the specified compressive strength, f'_m. The compressive stress shall also be limited by the maximum reinforcement ratio.

8.12.6.2: Nominal dimension of the column parallel to the plane of the wall frame shall not be less than two full units or 32 in. (810 mm), whichever is greater.

8.12.6.3: Nominal dimension of the column perpendicular to the plane of the wall frame shall not be less than 8 in. (203 mm) nor 1/14 of the clear height between beam faces.

8.12.6.4 Longitudinal Reinforcement:

8.12.6.4.1: A minimum of 4 longitudinal bars shall be provided at all sections of every wall frame column member.

8.12.6.4.2: The flexural reinforcement shall be essentially uniformly distributed across the member depth.

8.12.6.4.3: The nominal moment strength at any section along a member shall be not less than 1.6 times the cracking moment strength and the minimum reinforcement ratio shall be $130/f_y$ (the metric equivalent is $0.90/f_y$ where f_y is in MPa).

8.12.6.5 Transverse Reinforcement:

8.12.6.5.1: Transverse reinforcement shall be hooked around the extreme longitudinal bars and shall be terminated with a standard 180-degree hook.

8.12.6.5.2: The spacing of transverse reinforcement shall not exceed 1/4 the nominal dimension of the column parallel to the plane of the wall frame.

8.12.6.5.3: Minimum transverse reinforcement ratio shall be 0.0015.

8.12.7 WALL FRAME BEAM-COLUMN INTERSECTION:

8.12.7.1: Beam-column intersection dimensions in masonry wall frames shall be proportioned such that the wall frame column depth in the plane of the frame satisfies Eq. 8.12.7.1-1:

$$h_p > \frac{4{,}800\, d_{bb}}{\sqrt{f_g'}} \tag{8.12.7.1-1}$$

where:

h_p = pier depth in the plane of the wall frame, in.;

d_{bb} = diameter of the largest beam longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, in.; and

f_g' = specified compressive strength of grout, psi.

The metric equivalent of Eq. 8.12.7.1-1 is the same except that h_p and d_{bb} are in mm and f_g' is in MPa.

Beam depth in the plane of the frame shall satisfy Eq. 8.1.2.7.1-2:

$$h_b > \frac{4{,}800\, d_{bp}}{\sqrt{f_g'}} \tag{8.12.7.1-2}$$

where:

h_b = beam depth in the plane of the wall frame, in.;

d_{bp} = diameter of the largest column (pier) longitudinal reinforcing bar passing through, or anchored in, the wall frame beam-column intersection, in.; and

f_g' = specified compressive strength of grout, psi.

The metric equivalent of Eq. 8.12.7.1-2 is the same except that h_b and d_{bp} are in mm and f_g' is in MPa.

Nominal shear strength of beam-column intersections shall exceed the shear occurring when wall frame beams develop their nominal flexural strength.

8.12.7.2: Beam longitudinal reinforcement terminating in a wall frame column shall be extended to the far face of the column and shall be anchored by a standard 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:

$$A_s \geq \frac{0.5V_n}{f_y} \tag{8.12.7.2}$$

where:

A_s = cross-sectional area of reinforcement in.^{2};

V_n = nominal shear strength, lb; and

f_y = specified yield strength of the reinforcement or the anchor bolt as applicable, psi.

The metric equivalent of Eq. 8.12.7.2 is the same except that A_s is in mm^2, V_n is in N, and f_y is in MPa.

Special horizontal shear reinforcement shall be anchored by a standard hook around the extreme wall frame column reinforcing bars.

The nominal horizontal shear stress at the beam-column intersection shall not exceed the lesser of 350 psi (2.5 MPa) or $7\sqrt{f_m'}$ (the metric equivalent is 0.58 MPa $\sqrt{f_m'}$).

Appendix to Chapter 8

ALTERNATIVE MASONRY STRUCTURE DESIGN PROVISIONS

8A.1 GENERAL:

8A.1.1 SCOPE: The alternative provisions of this chapter for the design and construction of reinforced and plain (unreinforced) masonry components and systems for the materials used shall be permitted as an alternative design method for masonry designed by the allowable stress design method with appropriate modifications stated herein.

8A.1.2 REFERENCE DOCUMENT: The design, construction, and quality assurance of masonry components designed in accordance with the alternative masonry provisions of this chapter shall conform to the requirements of Ref. 8A-1 except as modified by the provisions of this appendix.

Ref. 8A-1 Building Code Requirements for Masonry Structures ACI 530-92/ASCE 5-92/TMS 402-92, including Appendix A, Special Provisions for Seismic Design, and Specifications for Masonry Structures, ACI 530.1-92/ASCE 6-92/TMS 602-92.

8A.1.2.1 MODIFICATIONS TO APPENDIX A OF REF. 8A-1:

8A.1.2.1.1: Replace all references to seismic zones (Ref. 8A-1) with the Seismic Performance Categories listed in Table 8A.1.2.1.1.

TABLE 8A.1.2.1.1
Ref. 8A-1 Appendix A Seismic Zones and Replacement Seismic Performance Categories

Appendix A (Ref.8A-1) Seismic Zone	Replace with Seismic Performance Category
0 and 1	A and B
2	C
3 and 4	D and E

8A.1.2.1.2: The requirements of Sec. 2.2.6 shall apply in lieu of the load and load combination provisions of Ref. 8A-1, Chapter 5.

8A.1.2.1.3: The requirements of Ref. 8A-1, Sec. A.3.3, shall not apply.

8A.1.2.1.4: The requirements of Ref. 8A-1, Sec. A.3.4, shall not apply.

8A.1.2.1.5: The requirements of Sec. 2.2.5.1.2 shall apply in lieu of the requirements of Ref. 8A-1, Sec. A.3.6.

8A.1.2.1.6: The requirements of Ref. 8A-1, Sec. A.4.9.1, shall not apply.

8A.1.2.1.7: The maximum spacing of reinforcement requirements of Sec. 8A.7.2.1 shall apply in lieu of those in Ref. 8A-1, Sec. A.4.9.1.2.

8A.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 strength reduction factor, ϕ, and 2.5 times the allowable working stress determined from Ref. 8A-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)	$\phi = 0.4$

8A.3 RESPONSE MODIFICATION COEFFICIENTS: The response modification coefficients, *R*, of Table 2.2.2 for reinforced masonry shall apply, provided masonry is designed in accordance with Ref. 8A-1, Chapter 7 and Appendix A. The *R* coefficients of Table 2.2.2 for unreinforced masonry shall apply for all other masonry.

8A.4 SEISMIC PERFORMANCE CATEGORY A: Buildings assigned to Category A may be of any type of masonry construction permitted by Ref. 8A-1.

8A.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. 8A-1, Chapter 6 or 7.

8A.6 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C shall conform to the requirements of Category B, to the requirements of Ref. 8A-1, Appendix A, and to the additional requirements of this section.

8A.6.1 CONSTRUCTION REQUIREMENTS:

8A.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. 8A-1, the other wythe shall be tied to its backup and reinforced with a minimum of one W1.7, (No. 9 wire gage) per nominal 4 in. (102 mm) of wythe or less at a maximum spacing of 16 in. (406 mm) on center. Wythe shall be tied in accordance with Ref. 8A-1, Sec. 5.8.2.2.

8A.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:

8A.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 reinforcement is continuous without offset. This continuous joint shall be reinforced with joint reinforcement having a minimum steel area of 0.03 in.^{2} (19 mm^2). Joint reinforcement shall be embedded in mortar or grout.

8A.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 in. (13 mm) of mortar cover.

8A.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 Non-load-bearing Wall Tile (ASTM C 56)

8A.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.

8A.7.1 CONSTRUCTION REQUIREMENTS FOR MASONRY LAID IN OTHER THAN RUNNING BOND: The maximum spacing of horizontal reinforcement shall not exceed 24 in. (610 mm).

8A.7.2 SHEAR WALL REQUIREMENTS: Shear walls shall comply with the requirements of this section.

8A.7.2.1: The maximum spacing of reinforcement in each direction shall be the smaller of the following dimensions: 1/3 the length and height of the element but not more than 48 in. (1220 mm). The area of reinforcement perpendicular to the shear reinforcement shall be at least equal to 1/3 the area of the required shear reinforcement. The portion of the reinforcement required to resist shear shall be uniformly distributed.

8A.7.2.2: When reinforcement is required in accordance with Ref. 8A-1, Sec. 7.5.2, the computed reinforcement shall be placed horizontally.

8A.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.

8A.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 reinforcement shall be a minimum of 1/2 in. (13 mm) for structural masonry.

8A.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.

8A.8.1.2 Stack Bond Construction: Masonry laid in stack bond shall conform to the following requirements:

8A.8.1.2.1: For masonry that is not part of the basic structural system, the minimum ratio of horizontal reinforcement shall be 0.0015 and the maximum spacing of horizontal reinforcement shall be 24 in (610 mm). For masonry that is part of the basic structural system, the minimum ratio of horizontal reinforcement shall be 0.0025 and the maximum spacing of horizontal reinforcement shall be 16 in. (406 mm). For the purpose of calculating this ratio, the area of joint reinforcement shall not be considered.

8A.8.1.2.2: Reinforced hollow unit construction shall be grouted solid and all head joints shall be made solid by the use of open end units.

Chapter 9

WOOD STRUCTURE DESIGN REQUIREMENTS

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 Wood Construction*, including Supplement	NFoPA (1991)
Ref. 9-2	*American Softwood Lumber Standard*	PS 20-70 (1986)
Ref. 9-3	*Softwood Plywood--Construction and Industrial*	PS 1-83 (1983)
Ref. 9-4	*Wood Particleboard*	ANSI A208.1 (1989)
Ref. 9-5	*Preservative Treatment by Pressure Process*	AWPA C1(1991), C2 and C3 (1991), C9 (1990), and C28 (1991)
Ref. 9-6	*American National Standard for Wood Products--Structural Glued Laminated Timber*	ANSI/AITC A190.1 (1992)
Ref. 9-7	*Wood Poles*	ANSI 05.1 (1992)
Ref. 9-8	*One- and Two-Family Dwelling Code*	Council of American Building Officials (CABO)
Ref. 9-9	*Plywood Design Specifications*	APA (1990)
Ref. 9-10	*Diaphragms*, Research Report 138	APA (1991)
Ref. 9-11	*Performance Standard for Wood-Based Structural-Use Panels*	PS2-92 (1992)
Ref. 9-12	*Design Capacities of APA Performance-Rated Structural-Use Panels*	APA N375A (1991)
Ref. 9-13	*Span Tables for Joists and Rafters*	NFoPA T903 (1992)
Ref. 9-14	*Standard Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists*	ASTM D5055 (1994)

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 sheet sheathing listed in Sec. 9.9.1.1 and 9.9.1.2 shall have nominal sheet sizes of 4 ft by 8 ft (1220 mm by 2440 mm) 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 2.16 times the stresses permitted in the reference documents and in this chapter. The value of the capacity reduction factor, ϕ, shall be as follows:

Wood members in flexure	$\phi = 1.00$
In compression	$\phi = 0.90$
In tension	$\phi = 1.00$
In shear and torsion	$\phi = 1.00$
Connectors	
Anchor bolts, bolts, lag bolts, nails, screws, etc.	$\phi = 0.85$
Bolts in single shear in members that are part of the seismic force resisting system	$\phi = 0.40$
Shear on diaphragms and shear walls (as given in this chapter)	$\phi = 0.60$

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 only conform to the requirements of Sec. 2.2.5.1.1. Buildings constructed in compliance with Sec. 9.10 are assumed to satisfy Sec. 2.2.5.1.1.

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, CONVENTIONAL CONSTRUCTION: Buildings not exempted by Sec. 1.2, which meet the height limits and braced wall spacing requirements of Table 9.10.1-1a may be constructed in conformance with the provisions of Sec. 9.10 or such buildings and all other buildings shall be designed in conformance with Sec. 9.8.

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 manufactured with intermediate or exterior glue.

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

9.5.2.1 Anchorage of Concrete or Masonry Walls: Ties and splices required in Sec. 2.2.5.1.1 and 2.2.5.1.2 shall be provided in Seismic Hazard Exposure Group III buildings with C_a equal to or greater than 0.10 and the diaphragm sheathing shall not be considered for this purpose.

9.5.2.2 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 FRAMING SYSTEMS: The limitations on framing systems that may be used in Category D construction are given below.

9.6.1.1 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.

9.6.1.2 Anchorage of Concrete and Masonry Walls: Ties and splices required in Sec. 2.2.5.1.1 and 2.2.5.1.2 shall be provided and the diaphragm sheathing shall not be considered for this purpose.

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

9.7.1 FRAMING SYSTEMS: Framing shall be designed in conformance with Sec. 9.8. Unblocked structural-use panel sheathed diaphragms shall not be considered to be part of the seismic force resisting system.

9.7.2 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 9.9.1-1b.

9.8 ENGINEERED WOOD CONSTRUCTION: Engineered wood construction is a structural system that is designed in conformance with well established principles of mechanics. When seismic analysis is prescribed for buildings, the proportioning and design of wood systems, members, and connections shall be in accordance with the reference documents and this section.

9.8.1 FRAMING REQUIREMENTS: All wood columns and posts shall be framed to true end bearing. Supports for columns and posts shall be designed to hold them securely in position and to provide protection against deterioration. Positive connections shall be provided to resist uplift and lateral displacement.

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

9.8.2.1 Framing: All framing used for shear panel construction shall conform to Ref. 9-2 for 2-by (actual 1.5 in., 38.1 mm) 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.

9.8.2.2 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.

9.8.2.3 Torsion: Buildings with two lines of resistance in either orthogonal direction 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:

Diaphragm sheathing shall conform to Sec. 9.9.1.1 through Sec 9.9.1.4. The width of the diaphragm normal to the orthogonal axis about which the torsional irregularity exists shall not exceed 25 ft (7620 mm) nor shall the l/w ratio be less than 1/1 for one-story buildings or 1.5/1 for buildings over one story in height, where l = the length of a diaphragm and w = the width of a diaphragm.

> **EXCEPTION:** Where calculations demonstrate that the diaphragm deflections can be tolerated, the width may be increased and the l/w ratio may be decreased to 1/1.5 when sheathed in conformance with Sec. 9.9.1.1 or to 1/1 when sheathed in conformance with Sec. 9.9.1.3 or 9.9.1.4.

9.9 DIAPHRAGMS AND SHEAR WALLS: The width of a panel sheet in a diaphragm or shear wall shall not be less than 2 ft (609.6 mm) and the l/w ratio of a diaphragm or h/w ratio of a shear wall shall not be more than permitted in Sec. 9.9.1.1 through 9.9.1.4 where: l = the length of a diaphragm, h = the height of a shear wall, w = the width of a diaphragm or shear wall, and h = the sum of the story heights for single element cantilevers or the height of the opening in pierced walls when detailed in conformance with Sec. 9.8.2.1. The h/w ratio shall not be greater than 2.1.

Capacities of diaphragms and shear walls may be calculated by principles of mechanics without limitation by using values of fastener strength and sheathing shear strength given in the reference standards. Fastener strength in the sheathing material must be based on verified test data from cyclical tests.

9.9.1 SHEAR PANEL REQUIREMENTS: Shear panels in diaphragms and shear walls shall conform to the requirements in this section. All panel sheathing joints in shear walls shall occur over studs or blocking. Where designated as blocked in Table 9.9.1-1a, all joints shall occur over framing members of the width prescribed in the table. Fasteners shall be placed at least 3/8 in. (9.53 mm) from ends of boards or edges of sheets.

The shear values for shear panels of different materials applied to the same wall line are not cumulative. The shear values for the same material applied to both faces of the same wall are cumulative. Adhesive attachment of wall sheathing is not permitted.

Sheet type sheathing shall be so arranged at boundaries and changes in direction of framing that no sheet has a minimum dimension less than 2 ft (609.6 mm).

9.9.1.1 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 9.9.1-1a for horizontal diaphragms and Table 9.9.1-1b for shear walls.

The edges of all structural-use panel sheets shall be supported by framing or blocking having the minimum width given in the tables for shear walls and blocked diaphragms. 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 9.9.1-1a and 9.9.1-1b. The *l/w* ratio and the *h/w* ratio shall not exceed the limits prescribed in Tables 9.9.1-1a and 9.9.1-1b.

9.9.1.2 Shear Panels Sheathed with Other Sheet Materials: Sheet materials other than structural-use materials have no recognized capacity for seismic force resistance and are not permitted as part of the seismic force resisting system except in conventional construction, Sec. 9.10.

9.9.1.3 Single Diagonally Sheathed Shear Panels: Single diagonally sheathed shear panels shall consist of 1-by (actual 3/4 in., 19.1 mm) sheathing boards laid at an angle of approximately 45 degrees (0.8 rad) to supports. Common nails at each intermediate support shall be two 8d for 1 by 6 (actual 3/4 in., 19.1 mm, by 5-1/2 in., 139.7 mm) and three 8d for 1 by 8 (actual 3/4 in., 19.1 mm, by 7-1/2 in., 190.5 mm) 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 2-by (actual 1-1/2 in., 38 mm) sheathing boards where 16d nails are substituted for 8d nails, end joints are located as above, and the support is not less than 3 in. (actual 2-1/2 in., 63.5 mm) width or 4 in. (actual 3-1/2 in., 88.9 mm) depth.

The allowable working stress shear for these panels is 200 plf (3000 kg/m). The *l/w* ratio shall not be more than 3/1 and the *h/w* ratio shall not be more than 2/1.

9.9.1.4 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 and for flexure between supports due to a uniform load equal to 50 percent of the shear per foot in the shear panel. The allowable working stress shear for these panels is 600 plf (8760 kg/m). The *l/w* ratio shall not be more than 3/1 and the *h/w* ratio shall not be more than 2/1.

9.10 CONVENTIONAL LIGHT FRAME CONSTRUCTION: Conventional light frame construction is a system of repetitive horizontal and vertical framing members selected from tables in Ref. 9-13 and conforming to the framing and bracing requirements of Ref. 9-8 except as modified by the provisions in this section. This system is limited to buildings with bearing wall heights not exceeding 10 ft and the number of stories prescribed in Table 9.10.1-1a. The gravity dead load of the construction is limited to 15 psf (718 Pa) for roofs and exterior walls and 10 psf (479 Pa) for floors and partitions and the gravity live load is limited to 40 psf (1915 Pa).

> **EXCEPTION:** Masonry veneer may be used on Category A buildings and on one-story Category B buildings.

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

9.10.1.1 Braced Wall Spacing: Braced exterior walls and braced partitions shall be located at the spacing indicated in Table 9.10.1-1a.

9.10.1.2 Braced Wall Sheathing Requirements: All braced walls and partitions shall be effectively and thoroughly braced by one of the types of sheathing prescribed in Table 9.10.1-1b. The length of bracing at each braced line is prescribed in Table 9.10.1-1b. 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 in. (1219.2 mm) in width covering three 16 in. (406.4 mm) stud spaces or two 24 in. (609.6 mm) stud spaces for diagonal boards or structural-use panel sheets and shall be at least 96 in. (2438.4 mm) in width covering six 16-in. (406.4 mm) stud spaces or four 24 in. (609.6 mm) stud spaces for all other sheathing. All panel sheathing joints shall occur over studs or blocking. Sheathing shall be fastened to all studs, plates, and at panel edges occurring over blocking. All wall framing to which sheathing used for bracing is applied shall conform to Ref. 9-2 for 2 (actual 1-1/2 inch, 38.1 mm) by or larger members.

Panel sheathing nailing shall be not less than the minimum given in Table 9.9.1-1b or as prescribed in Table 9.10.1-1b. Nailing for diagonal boards shall be as prescribed in Sec. 9.9.1.3 and 9.9.1.4. Adhesive attachment of wall sheathing is not permitted.

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 capacity of the braced panels at exterior cripple stud walls shall be increased to compensate for length of interior braced wall eliminated by increasing the length of the sheathing or increasing the number of fasteners.

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

9.10.2.1 Wall Anchorage: Anchorage for braced wall sills to concrete or masonry foundations conforming to the requirements of Chapters 6 and 8 shall be provided. Such anchorage may be provided by 1/2 in. (12.7 mm) diameter anchor bolts having a minimum embedment of 7 bolt diameters spaced at not over 6 ft (1828.2 mm) on center for one- and two-story buildings and at not more than 4 ft (1219.2 mm) on center for buildings over two stories in height. Other anchorage devices having equivalent capacity are permitted.

9.10.2.2 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 ft (1219.2 mm). Single top plates may be used when they are spliced by framing devices providing capacity equivalent to the lapped splice prescribed for double top plates.

9.10.2.3 Bottom Plates: Studs shall have full bearing on a plate or sill conforming to Ref. 9-2 for 2-by (actual 1-1/2 in., 38.1 mm) or larger members having a width at least equal to the width of the studs.

9.10.2.4 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 with toe nailing using three 8d nails per joist or rafter where these elements are spaced at not over 2 feet (609.6 mm) on center or 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 ft (15.24 m) 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.

TABLE 9.9.1-1a
Allowable Shear in Pounds per Foot (at Working Stress) for Horizontal Wood Diaphragms with Framing Members of Douglas Fir, Larch, or Southern Pine for Seismic Loading[a]

Panel Grade	Fastener[b]		Minimum nominal panel thickness (in.)	Minimum nominal width of framing (in.)	Lines of fasteners	Blocked Diaphragms							Unblocked Diaphragms[c]	
	Type	Minimum penetration in framing (in.)				Fastener spacing at diaphragm boundaries (all cases, at continuous panel edges parallel to load (Cases 3 and 4) and at all panel edges (Cases 5 and 6)[d]							Fastener spacing at 6 in. centers at supported edges	
						6	4		2-1/2[e]		2[e]			
						Spacing per line at other panel edges (in.)							Case 1	Cases 2, 3, 4, 5, and 6
						6	6	4	4	3	3	2		
Structural I	6d common	1-1/4	5/16	2 3	1 1	185 210	250 280	--- ---	375 420	--- ---	420 475	--- ---	165 185	125 140
	8d common	1-1/2	3/8	2 3	1 1	270 300	360 400	--- ---	530 600	--- ---	600 675	--- ---	240 265	180 200
	10d common	1-5/8	15/32	2 3	1 1	320 360	425 480	--- ---	640 720	--- ---	730 820	--- ---	285 320	215 240
	10d common	1-5/8	23/32	3 4 4	2 2 3	--- --- ---	650 755 940	870 980 1305	940 1080 1375	1230 1410 1800	--- --- ---	--- --- ---	--- --- ---	--- --- ---
	14 gauge staples	2	23/32	3 4	2 3	--- ---	600 840	600 900	840 1140	900 1350	1040 1440	1200 1800	--- ---	--- ---
Sheathing, single floor and other grades covered in Ref. 9-3 and 9-13	6d common	1-1/4	5/16	2 3	1 1	170 190	225 250	--- ---	335 380	--- ---	380 430	--- ---	150 170	110 125
			3/8	2 3	1 1	185 210	250 280	--- ---	375 420	--- ---	420 475	--- ---	175 185	125 140
	8d common	1-1/2	3/8	2 3	1 1	240 270	320 360	--- ---	480 540	--- ---	545 610	--- ---	215 240	160 180
			7/16	2 3	1 1	255 285	340 380	--- ---	575 645	--- ---	575 645	--- ---	230 255	170 190

Sheathing, single floor and other grades covered in Ref. 9-3 and 9-13 cont.	10d common	1-5/8	15/32	2	1	290	385	---	575	---	655	---	255	190
				3	1	325	430	---	650	---	735	---	290	215
			19/32	2	1	320	425	---	640	---	730	---	285	215
				3	1	360	480	---	720	---	820	---	320	240
			23/32	3	2		645	870	935	1225	---	---	---	---
				4	2		750	980	1075	1395	---	---	---	---
				4	3		935	1305	1390	1510	---	---	---	---
	14 gauge staples	2	23/32	3	2		600	600	820	900	1020	1200	---	---
				4	3		820	900	1120	1350	1400	1510	---	---

NOTE: For conversion to metric, 1 in. = 25.4 mm and 1 ft = 305 mm.

[a] l/w shall not be more than 4/1 for blocked diaphragms or more than 3/1 for unblocked diaphragms. For framing members of other species set forth in Ref 9-1, Table 12A, with the range of specific gravity (SG) noted, allowable shear values shall be calculated for all grades by multiplying the values for Structural I or panel siding, as appropriate, by the following factors: 0.82 for SG equal to or greater than 0.42 but less than 0.49 ($0.42 = SG < 0.49$) and 0.65 for SG less than 0.42 ($SG < 0.42$).

[b] Space nails along intermediate framing members at 12-in. centers except where spans are greater than 32 in.; space nails at 6-in. centers.

[c] Blocked values may be used for 1-1/8-in. panels with tongue-and-groove edges where 1 in. by 3/8 in. crown by No. 16 gauge staples are driven through the tongue-and-groove edges 3/8 in. 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.

[d] Maximum shear for Cases 3 through 6 is limited to 1200 pounds per foot.

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

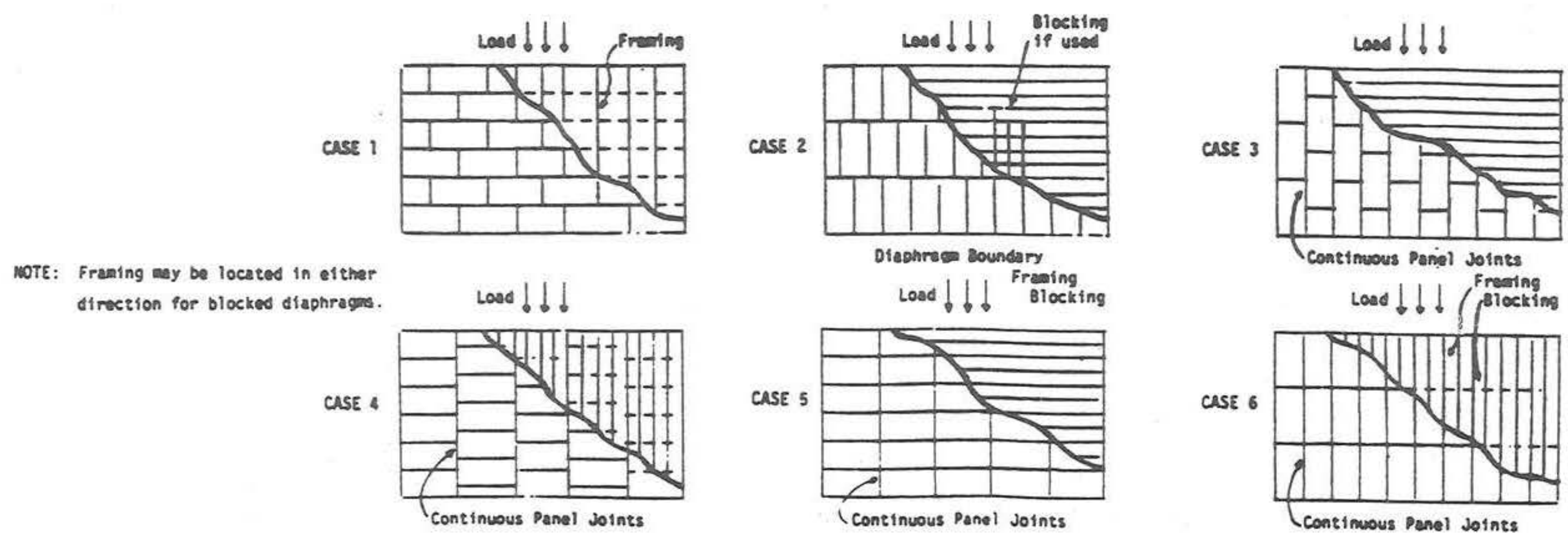

TABLE 9.9.1-1b
Allowable Working Stress Shear in Pounds per Foot for Seismic Forces for Structural-Use Panel Shear Walls with Framing of Douglas Fir, Larch, or Southern Pine[a]

Panel Grade	Nail Size[b]	Penetration in Framing (in.)	Panel Thickness (in.)	Panel Applied Direct to Framing[c] 6	4	3[d]	2[d]	Nail Size[b]	Panel Applied Over 1/2-in. Gypsum Sheathing[c] 6	4	3[d]	2[d]
Structural I	6d	1-1/4	5/16	200	300	390	610	8d	200	300	390	510
	8d	1-1/2	3/8[e]	230	360	460	610	10d	230	360	460	610
	8d	1-1/2	15/32	280	430	550	730	10d	280	430	550	730
	10d	1-5/8	15/32	340	510	665	870	---	---	---	---	---
All other sheathing grades covered in Ref. 9-3 and 9-13	6d	1-1/4	5/16	180	270	350	450	8d	180	270	350	450
	6d	1-1/4	3/8	200	300	390	510	8d	200	300	390	510
	8d	1-1/2	3/8[e]	220	320	410	530	10d	220	320	410	530
	8d	1-1/2	15/32	260	380	490	640	10d	260	380	490	640
	10d	1-5/8	15/32	310	460	600	770	---	---	---	---	---
	10d	1-5/8	15/32	340	510	665	870	---	---	---	---	---
Panel siding in grades covered in Ref. 9-3	6d	1-1/4	5/16	140	210	275	360	8d	140	210	275	360
	8d	1-1/2	3/8[e]	160	240	310	410	10d	160	240	310	410

NOTE: For conversion to metric, 1 in. = 25.4 mm and 1 ft = 305 mm.

[a] *h/w* ratio shall not be more than 2/1. All panel edges backed with 2-in. nominal or wider framing. Panels installed either horizontally or vertically. Space nails at 6 in. on center along intermediate framing members for 3/8-in. panels installed with face grain parallel to studs spaced 24 in. on center and 12 in. on center for other conditions and panel thicknesses. For framing members of other species set forth in Ref. 9.1, Table 12A, with the range of specific gravity (SG) noted, allowable shear values shall be calculated for all grades using common or galvanized box nails by multiplying the values for Structural I by the following factors: 0.82 for SG equal to or greater than 0.42 but less than 0.49 ($0.42 = SG < 0.49$) and 0.65 for SG less than 0.42 ($SG < 0.42$). For panel siding using galvanized casing nails, the allowable shear values shall be the values in the table for panel siding multiplied by the same factors.

[b] Nails for values shown for Structural I and all other sheathing grades shall be common nails or galvanized box nails and for values shown for panel siding shall be galvanized casing nails.

[c] Nail spacing at panel edges.

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

[e] The values for 3/8-in. panels applied directly to framing may be increased 20 percent provided studs are a minimum of 16 in. on center or panel is applied with face grain across studs.

TABLE 9.10.1-1a
Conventional Construction Braced Wall Requirements

Seismic Performance Category	Maximum Distance Between Braced Walls	Maximum Number of Stories Permitted [a]
A[b]	35 ft	3
B	35 ft	3
C	25 ft	2
D	25 ft	1 [c]
E	Conventional construction not permitted; conformance with Sec. 9.8 required.	

NOTE: For conversion to metric, 1 ft = 305 mm.

[a] A cripple wall is considered to be a story. Maximum bearing wall height shall not exceed 10 ft.

[b] See exceptions to Sec. 1.2.

[c] Detached one- and two-family dwellings may be two stories.

TABLE 9.10.1-1b
Conventional Construction Braced Wall Requirements in Minimum Length of Wall Bracing per 25 Lineal Feet of Braced Wall Line[a]

Story Location	Sheathing Type[b]	$0.05 \le C_a < 0.10$	$0.10 \le C_a < 0.15$	$0.15 \le C_a < 0.20$	$0.20 \le C_a < 0.30$	$0.30 \le C_a \le 0.40$[e]
Top or only story	G-P[d]	8 ft 0 in.	8 ft 0 in.	10 ft 8 in.	14 ft 8 in.	18 ft 8 in.[c]
	S-W	4 ft 0 in.	4 ft 0 in.	5 ft 4 in.	8 ft 0 in.	9 ft 4 in.[c]
Story below top story	G-P[d]	10 ft 8 in.	14 ft 8 in.	18 ft. 8 in.[c]	NP	NP
	S-W	5 ft 4 in.	6 ft 8 in.	10 ft 8 in.[c]	13 ft 4 in.[c]	17 ft 4 in.[c]
Bottom story of 3 stories	G-P[d]	14 ft 8 in.	Conventional construction not permitted; conformance with Sec. 9.8 required.			
	S-W	8 ft 0 in.				

NOTE: For conversion to metric, 1 in. = 25.4 mm and 1 ft = 305 mm.

[a] Minimum length of panel bracing of one face of wall for S-W sheathing or both faces of wall for G-P sheathing; *h/w* ratio shall not exceed 2/1. For S-W panel bracing of the same material on two faces of the wall, the minimum length may be one half the tabulated value but the *h/w* ratio shall not exceed 2/1 and design for uplift is required.

[b] G-P = gypsumboard, particleboard, lath and plaster, or gypsum sheathing boards; S-W structural-use panels and diagonal wood sheathing. NP = not permitted.

[c] Applies to one- and two-family detached dwellings only.

[d] Nailing shall be as follows:

For 1/2-in. gypsum board, 5d (0.113 in. diameter) coolers at 7 in. centers;

For 5/8-in. gypsum board, No. 11 gauge (0.120 in. diameter);

For gypsum sheathing board, 1-3/4-in. long by 7/116-in. head, diamond point galvanized at 4 in. centers;

For gypsum lath, No. 13 gauge (0.092 in.) by 1-1/8-in. long, 19/64 head, plasterboard at 5 in. centers;

For Portland cement plaster, No. 11 gauge by 1-1/2-in. long, 7/16 head at 6 in. centers;

For fiberboard, No. 11 gauge by 1-1/2-in. long, 7/16 head, galvanized at 3 in. centers.

Nailing as specified above shall occur at all panel edges at studs, at top and bottom plates, and, where occurring, at blocking.

[e] Where $C_a > 0.40$, conventional construction is not permitted.

GLOSSARY

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 Component Support: Those structural members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between architectural systems, components, or elements and the building structure.

Attachments: Means by which components and their supports are secured or connected to the seismic-force-resisting system of the structure. Such attachments include anchor bolts, welded connections, and mechanical fasteners.

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.

Brittle: Systems, members, materials, and connections that do not exhibit significant energy dissipation capacity in the inelastic range.

Building Structure: Self-supporting structure, at grade, whose primary function includes shelter of human occupants.

Boundary Members: Portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement and/or structural steel members.

Collector Elements: Members that serve to transfer forces between floor diaphragms and members of the lateral-force-resisting system.

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

Component, Equipment: A mechanical or electrical component or element that is part of a mechanical and/or electrical system within or without a building system.

Component, Flexible: Component, including its attachments, having a fundamental period greater than 0.06 sec.

Component, Rigid: Component, including its attachments, having a fundamental period less than or equal to 0.06 sec.

Composite (Chapter 7):

Composite Beam: An steel beam either fully encased in concrete or an unencased steel beam made to act integrally with a concrete or composite slab using shear connectors.

Composite Brace: A steel brace fabricated from rolled or built-up structural steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete.

Composite Column: A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete where the structural steel portion accounts for at least 4 percent of the gross column area.

Composite Slab: Slab system consisting of a concrete slab and deformed metal deck where the two act compositely in flexure and shear.

Composite Shear Walls: Walls consisting of steel plates with concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel panel.

Fully Composite Beam: A composite beam where the shear connectors are provided in sufficient numbers to develop the nominal plastic flexural strength of the composite section.

Partially Composite Beam: An unencased steel beam where the number and strength of shear connectors governs the flexural strength of the composite section.

Partially Restrained Composite Connection: Partially restrained connections as defined in Ref. 7-1 between partially or fully composite beams to steel columns where bending resistance is provided by a couple consisting of steel reinforcing bars in the slab and steel seat angles. The connections typically are designed to transmit moments less than the full capacity of the beams and columns.

Concrete

Plain Concrete: Concrete that is either unreinforced or contains less reinforcement than the minimum amount specified in Ref. 6-1 for reinforced concrete.

Reinforced Concrete: Concrete reinforced with no less than the minimum amount required by Ref. 6-1, prestressed or nonprestressed, and designed on the assumption that the two materials act together in resisting forces.

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.

Coupling Beam: A beam that is used to connect adjacent concrete wall piers to make them act together as a unit to resist lateral loads.

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 System: 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.

Displacement

Design Displacement: The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

Total Design Displacement: The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.

Total Maximum Displacement: The maximum capable earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of building separations, and vertical load testing of isolator unit prototypes.

Displacement Restraint System: A collection of structural elements that limits lateral displacement due to the maximum capable earthquake.

Effective Damping: The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.

Effective Stiffness: The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.

Encased Composite Beam: A beam totally encased in concrete cast integrally with the slab where full composite action is provided by bond between the steel and concrete without additional anchorage, Chapter 7.

Encased Shapes: Structural steel members that are encased in structural concrete, Chapter 7.

Enclosure: An interior space surrounded by walls.

Equipment Support: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers or saddles, that transmit gravity and operating loads between the equipment and the structure.

Face Bearing Plates: Stiffener plates that are attached to steel beams embedded in concrete walls or columns, Chapter 7. The plates are located at the face of the concrete to provide confinement and transfer forces to the concrete through direct bearing.

Filled Tubes: Structural steel tubes or pipes that are filled with structural concrete, Chapter 7.

Flexible Equipment Connections: Those connections between equipment components that permit rotational and/or translational movement without degradation of performance. Examples include universal joints, bellows expansion joints, and flexible metal hose.

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.

Ordinary Concentrically Braced Frame (OCBF): A steel concentrically braced frame in which members and connections are designed in accordance with the provisions of Ref. 5-3 without modification.

Special Concentrically Braced Frame (SCBF): A steel or composite steel and concrete concentrically braced frame in which members and connections are designed for ductile behavior. Special concentrically braced frames shall conform to Sec. 5.2.1 or 7.4.4.

Moment Frame:

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; see Sec. 6.3.2.

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; see Sec. 5.6, 6.3.1, 7.4.2, or 8.3.

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; see Sec. 5.6, 6.3.3, 7.4.3, or 8.12.

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. 2.2.2.1.

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.

Hazardous Contents: A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life-safety threat to the general public if an uncontrolled release were to occur.

High Temperature Energy Source: A fluid, gas, or vapor whose temperature exceeds 220 degrees F (378 K).

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 framing at the top.

Isolation Interface: The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

Isolation System: The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system if such a system is used to meet the design requirements of this section.

Isolator Unit: A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under design seismic load. An isolator unit may be used either as part of or in addition to the weight-supporting system of the building.

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

Load:

Dead Load: 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. 2.3.2.

Live Load: The load superimposed by the use and occupancy of the building not including the wind load, earthquake load, or dead load; see Sec. 2.3.2.

Load Carrying Bars: Reinforcing bars in composite members that are designed and detailed to resist calculated forces, Chapter 7.

Maximum Capable Earthquake: The maximum level of earthquake ground shaking that may ever be expected at the building site within the known geological framework. In map areas with an A_a value of 0.3 or greater, this intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 100-year time period.

Nonbuilding Structure: Self-supporting structure, at grade, whose primary function does not include shelter of human occupants.

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.

Restraining Bars: Reinforcing bars in composite members that are provided primarily to facilitate erection of the reinforcement and are not designed to carry calculated forces, Chapter 7. Generally, such bars are not spliced to be continuous.

Roofing Unit: A unit of roofing material weighing more than 1 pound (0.5 kg).

Seismic Coefficients: Coefficients C_a and C_v determined from Sec. 1.4.2.3 or Tables 1.4.2.4a or 1.4.2.4b based on Soil Profile Type and A_a and A_v, respectively.

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. 1.4.3.

Seismic Performance Category: A classification assigned to a building as defined in Sec. 1.4.4.

Seismic Response Coefficient: Coefficient C_s as determined from Sec. 2.3.2.1.

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

Site Coefficients: The values of F_a and F_v indicated in Tables 1.4.2.3a and 1.4.2.3b, respectively.

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.

Storage Racks: Include industrial pallet racks, movable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

Story Drift Ratio: The story drift, as determined in Sec. 2.3.7, divided by the story height.

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

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 the provisions and assumptions of the strength design methods of these provisions (or the referenced standards) before application of any strength reduction factors.

Required Strength: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by these provisions.

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.

Toughness: The ability of a material to absorb energy without losing significant strength.

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 that has a slope of 60 degrees or greater with the horizontal plane 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 (350 mm) 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.

Wind-Restraint System: The collection of structural elements that provides restraint of the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of isolator units or a separate device.

NOTATIONS

A, B, C, D, E, F	The Soil Profile Types as defined in Sec. 1.4.2.
A_a	The effective peak acceleration coefficient as determined in Sec. 1.4.1.
A_b	Area (in.^{2} or mm^2) of anchor bolt or stud in Chapters 6 and 8.
A_{ch}	Cross-sectional area (in.^{2} or mm^2) of a component measured to the outside of the special lateral reinforcement.
A_g	The component acceleration coefficient (expressed as a percentage of gravity) at base of structure (ground), as defined in Sec. 3.1.3.
A_n	Net cross-sectional area of masonry (in.^{2} or mm^2) in Chapter 8.
A_o	The area of the load-carrying foundation (ft^2 or m^2).
A_p	The component acceleration coefficient (expressed as a percentage of gravity) at point of attachment to structure as defined in Sec. 3.1.3.
A_p	The area of an assumed failure surface taken as a pyramid in Eq. 6.2.4.1-3 or in Chapter 6.
A_p	Projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations (in.^{2} or mm^2) in Chapter 8.
A_r	The component acceleration coefficient (expressed as a percentage of gravity) at structure roof level as defined in Sec. 3.1.3.
A_s	The structure response acceleration coefficient (expressed as a percentage of gravity) as defined in Sec. 3.1.3.
A_s	The area of an assumed failure surface taken as a pyramid in Eq. 6.2.4.1-3 or in Chapter 6.
A_s	Cross-sectional area of reinforcement (in.^{2} or mm^2) in Chapters 6 and 8.
A_s	Cross sectional area of structural steel elements in composite members in Chapter 7.

A_s/A_g	Ratio of cross sectional area of structural steel portion to the gross area of a composite column (see Ref. 7-1)
A_{sc}	Area of encased steel shape on the compression side of the plastic neutral axis in a composite column in Chapter 7.
A_{sh}	Total cross-sectional area of hoop reinforcement (in.^{2} or mm^2), including supplementary cross-ties, having a spacing of s_h and crossing a section with a core dimension of h_c.
A_{sp}	Horizontal area of the steel plate in composite shear wall (Eq. 7.4.7.1.1)
A_{st}	Area of encased steel shape on the tension side of the plastic neutral axis in a composite column in Chapter 7.
A_t	The area (in.^{2} or mm^2) of the flat bottom of the truncated pyramid of an assumed concrete failure surface in Sec. 6.2.4.1 or Eq. 6.2.4.1-3.
A_v	The effective peak velocity-related acceleration coefficient as determined in Sec. 1.4.1.
A_{vd}	Required area of leg (in.^{2} or mm^2) of diagonal reinforcement.
A_x	The torsional amplification factor.
a_b	Length of compressive stress block (in. or mm) in Chapter 8.
a_d	The incremental factor related to P-delta effects in Sec. 2.3.6.2.
a_p	The component amplification factor as defined in Sec. 3.1.3.
B_I	Numerical coefficient related to the effective damping of the isolation system as set forth in Table 2.6.3.3.1b.
B_a	Nominal axial strength of an anchor bolt (lb or N) in Chapter 8.
B_v	Nominal shear strength of an anchor bolt (lb or N) in Chapter 8.
b	Flange width of structural steel tubing in Chapter 7.
b_a	Factored axial force on an anchor bolt (lb or N) in Chapter 8.
b_p	The shortest plan dimension of the structure, in feet (mm), measured perpendicular to d_p (Sec. 2.6).
b_v	Factored shear force on an anchor bolt (lb or N) in Chapter 8.

b_w	Web width (in. or mm) in Chapter 8.
b_w	Effective width for shear strength calculation of reinforced concrete portion of composite column in Chapter 7.
b/t	Width to thickness ratio of either unstiffened or stiffened elements, as applicable, of steel members under compression stresses (see Ref. 7-1).
C_u	Coefficient for upper limit on calculated period; see Table 2.3.3.
C_a	Seismic coefficient determined from Sec. 1.4.2.3 or Table 1.4.2.4a based on Soil Profile Type and A_a.
C_d	The deflection amplification factor as given in Table 2.2.2.
C_s	The seismic response coefficient (dimensionless) determined in Sec. 2.3.
$\tilde{C}_s$	The seismic response coefficient (dimensionless) determined in Sec. 2.5.2.1 and 2.5.3.1.
C_{sm}	The modal seismic response coefficient (dimensionless) determined in Sec. 2.4.5.
C_T	The building period coefficient in Sec. 2.3.3.1.
C_v	Seismic coefficient determined from Sec. 1.4.2.3 or Table 1.4.2.4b based on Soil Profile Type and A_v.
C_{vx}	The vertical distribution factor as determined in Sec. 2.3.4.
c	Distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (in. or mm).
c_{eq}	Effective energy dissipation device damping coefficient (Eq. 2A.3.2.1).
D	The effect of dead load in Sec. 2.2.6.
D	Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.3.1.
D_p	Relative seismic displacement that the component must be designed to accommodate as defined in Sec. 3.1.4.
D/t	Diameter to wall thickness ratio of steel pipes used for concrete filled composite columns (see Ref. 7-1).

D'	Design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 2.6.3.4.2.
D_s	The total depth of the stratum in Eq. 2.5.2.1.2-4 (ft or m).
D_T	Total design displacement, in inches (mm), of an element of the isolation system including both translational displacement at the center of rigidity, D, and the component of torsional displacement in the direction under consideration as specified in Sec. 2.6.3.3.3.
D_{TM}	Total maximum displacement, in inches (mm), of an element of the isolation system including both translational displacement at the center of rigidity and the component of torsional displacement in the direction under consideration as prescribed by Eq. 2.6.3.3.4.
d	Overall depth of member (in. or mm) in Chapters 5 and 8.
d	Depth of steel beam (Eq. 7.5.2.2).
d_b	Diameter of reinforcement (in. or mm) in Chapter 8.
d_e	Distance from the anchor axis to the free edge (in. or mm) in Chapter 6.
d_p	The longest plan dimension of the structure, in feet (mm).
E	The effect of horizontal and vertical earthquake induced forces (Sec. 2.2.6).
E_m	Chord modulus of elasticity of masonry (psi or MPa) in Chapter 8.
E_s	Modulus of elasticity of reinforcement (psi or MPa) in Chapter 8.
E_s	Elastic modulus of structural steel in Chapter 7.
E_v	Modulus of rigidity of masonry (psi or MPa) in Chapter 8.
e_p	The actual eccentricity, in feet (mm), measured in plan between the center of mass of the structure above the isolation interface and the center of rigidity of the isolation system, plus accidental eccentricity, in feet (mm), taken as 5 percent of the maximum building dimension perpendicular to the direction of force under consideration.
F_a	Acceleration-based site coefficient (at 0.3 sec period).

F_I^-	Maximum negative force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ_I^-.
F_{max}^-	Maximum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ_I^-.
F^-_{min}	Minimum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ_I^-.
F^+	Maximum positive force in an isolator unit during a single cycle of prototype testing at a displacement amplitude of Δ_I^+.
F^+_{max}	Maximum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ_I^+.
F^+_{min}	Minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude of Δ_I^+.
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. 2.3.4 (kip or kN).
F_p	The seismic design force applied to a component at its center of gravity as determined in Sec. 3.1.3.
F_p	The induced seismic force on connections and anchorages as determined in Sec. 2.2.5.1.
F_u	Specified ultimate tensile strength (psi or MPa) of an anchor (Sec. 6.2.4).
F_v	Velocity-based site coefficient (at 1.0 sec period).
F_x	Total force distributed over the height of the structure above the isolation interface as prescribed by Eq. 2.6.3.5.
F_{xm}	The portion of the seismic base shear, V_m, induced at Level x as determined in Sec. 2.4.6 (kip or kN).
F_y	Specified yield strength of structural steel in Chapter 7.
F_{yh}	Specified yield strength of reinforcing bar ties in Chapter 7.
f_c'	Specified compressive strength of concrete used in design.
f_m'	Specified compressive strength of masonry (psi or MPa) at the age of 28 days unless a different age is specified, Chapter 8.

f_r	Modulus of rupture of masonry (psi or MPa) in Chapter 8.
f'_s	Ultimate tensile strength (psi or MPa) of the bolt, stud, or insert leg wires. For A307 bolts or A108 studs, may be assumed to be 60,000 psi (415 MPa).
f_y	Specified yield strength of reinforcement (psi or MPa).
f_{yh}	Specified yield stress of the special lateral reinforcement (psi or kPa).
G	$\gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels (psf or Pa).
G_o	$\gamma v_{so}^2/g$ = = the average shear modulus for the soils beneath the foundation at small strain levels (psf or Pa).
g	Acceleration of gravity in in./sec^2 (mm/s^2).
H	Thickness of soil.
h	The cross sectional dimension of reinforced concrete or composite columns in Chapter 7.
h_c	The cross sectional dimension of the confined core region in composite columns measured center-to-center of the tie reinforcement in Chapter 7.
$\bar{h}$	The effective height of the building as determined in Sec. 2.5.2 or 2.5.3 (ft or m).
h	Height of a wood shear panel or diaphragm (ft or mm) in Chapter 9.
h	The roof elevation of a structure in Chapter 3.
h	Height of the member between points of support (in. or mm) in Chapter 8.
h_c	The core dimension of a component measured to the outside of the special lateral reinforcement (in. or mm).
h_i, h_n, h_x	The height above the base Level i, n, or x, respectively (ft or m).
h_{sx}	The story height below Level $x = h_x - h_{x-1}$ (ft or m).
h/tw	Height to thickness ratio of web elements of steel members (see Ref. 7.1).

I_{cr}	Moment of inertia of the cracked section (in.^{4} or mm^4) in Chapter 8.
I_n	Moment of inertia of the net cross-sectional area of a member (in.^{4} or mm^4) in Chapter 8.
I_o	The static moment of inertia of the load-carrying foundation; see Sec. - 2.5.2.1 (in^4 or mm^4).
I_p	The component importance factor as prescribed in Sec. 3.1.5.
i	The building level referred to by the subscript i; i = 1 designates the first level above the base.
K_p	The stiffness of component or attachment as defined in Sec. 3.3.3.
K_y	The lateral stiffness of the foundation as defined in Sec. 2.5.2.1.1 (lb/in. or N/m).
K_θ	The rocking stiffness of the foundation as defined in Sec. 2.5.2.1.1 (ft·lb/degree or N·m/rad).
KL/r	The lateral slenderness of a compression member measured in terms of its effective buckling length, KL, and the least radius of gyration of the member cross section, r.
k	The distribution exponent given in Sec. 2.3.4
k_{eff}	Effective stiffness of an isolator unit, as prescribed by Eq. 2.6.9.3.
k_{max}	Maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.
k_{min}	Minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.
$\bar{k}$	The stiffness of the building as determined in Sec. 2.5.2.1.1 (lb/ft or N/m).
L	The overall length of the building (ft or m) at the base in the direction being analyzed.
L	Length of bracing member (in. or mm) in Chapter 5.
L	Length of coupling beam between coupled shear walls in Chapter 8 (in. or mm).

L_o	The overall length of the side of the foundation in the direction being analyzed, Sec. 2.5.2.1.2 (ft or m).
l	Length of a wood shear panel or diaphragm in Chapter 9 (ft or mm).
ℓ_b	Effective embedment length of anchor bolt (in. or mm) in Chapter 8.
ℓ_{be}	Anchor bolt edge distance (in. or mm) in Chapter 8.
ℓ_d	Development length (in. or mm) in Chapter 8.
ℓ_{dh}	Equivalent development length for a standard hook (in. or mm) in Chapter 8.
ℓ_{ld}	Minimum lap splice length (in. or mm) in Chapter 8.
M	Moment on a masonry section due to unfactored loads (in.·lb or N·mm) in Chapter 8.
M_a	Maximum moment in a member at stage deflection is computed (in.·lb or N·mm) in Chapter 8.
M_{cr}	Cracking moment strength of the masonry (in.·lb or N·mm) in the Chapter 8.
M_d	Design moment strength (in.·lb or N·mm) in Chapter 8.
M_f	The foundation overturning design moment as defined in Sec. 2.3.6 (ft·kip or kN·m).
M_o, M_{o1}	The overturning moment at the foundation-soil interface as determined in Sec. 2.5.2.3 and 2.5.3.2 (ft·lb or N·m).
M_M	Numerical coefficient related to maximum capable earthquake response as set forth in Table 2.6.3.3.4.
M_{nb}	Unfactored ultimate moment capacity at balanced strain conditions (Sec. 4.5.3.4).
M_t	The torsional moment resulting from the location of the building masses, Sec. 2.3.5.1 (ft·kip or kN·m).
M_{ta}	The accidental torsional moment as determined in Sec. 2.3.5.1 (ft·kip or kN·m).
M_u	Required flexural strength due to factored loads (in.·lb or N·mm) in Chapter 8.

M_1, M_2	Nominal moment strength at the ends of the coupling beam (in.·lb or N·mm) in Chapter 8.
M_x	The building overturning design moment at Level x as defined in Sec. 2.3.6 or Sec. 2.4.10 (ft·kip or kN·m).
m	A subscript denoting the mode of vibration under consideration; i.e., $m = 1$ for the fundamental mode.
N	Number of stories, Sec. 2.3.3.1.
N	Standard penetration resistance, ASTM D1536-84.
$\bar{N}$	Average field standard penetration test for the top 100 ft (30 m); see Sec. 1.4.2.
N_{ch}	Average standard penetration for cohesionless soil layers for the top 100 ft (30 m); see Sec. 1.4.2.
N_s	Numerical coefficient related to both the proximity of the building to an active fault and fault magnitude as set forth in Table 2.6.3.3.1a.
N_v	Force acting normal to shear surface (lb or N) in Chapter 8.
n	Designates the level that is uppermost in the main portion of the building.
n	Number of anchors (Sec. 6.2.4).
P	Axial load on a masonry section due to unfactored loads (lb or N) in Chapter 8.
P_c	Design tensile strength governed by concrete failure of anchor bolts (Sec. 6.2.4).
P_D	Required axial strength on a column resulting from application of dead load, D, in Chapter 5 (kip or kN).
P_E	Required axial strength on a column resulting from application of the amplified earthquake load, E', in Chapter 5 (kip or kN).
P_L	Required axial strength on a column resulting from application of live load, L, in Chapter 5 (kip or kN).
P_n	Nominal axial load strength (lb or N) in Chapter 5.

P_n	The algebraic sum of the seismic forces and the minimum gravity loads on the joint surface acting simultaneously with the shear (lb or N).
P_n	Nominal axial load strength (lb or N) in Chapter 8.
P_n	Nominal axial strength of column in Chapter 7.
P_s	Design tensile strength governed by steel of anchor bolts in Chapter 6.
P_u	Required axial design strength of column in Chapter 7.
P_u	Required axial load (lb or N) in Chapter 8.
P_u	Tensile strength required due to factored loads (lb or N) in Chapter 6.
P_u^*	Required axial strength on a brace (kip or kN) in Chapter 5.
P_x	The total unfactored vertical design load at and above Level x (kip or kN).
PI	Plasticity index, ASTM D4318-93.
Q_E	The effect of horizontal building forces (kip or kN); see Sec. 2.2.6.
Q_V	The load equivalent to the effect of the horizontal and vertical shear strength of the vertical segment, Appendix to Chapter 5.
R	The response modification coefficient as given in Table 2.2.2.
R_I	Numerical coefficient related to the type of lateral-force-resisting system above the isolation system as set forth in Table 2.6.3.4.2 for seismically isolated structures.
R_p	The component response modification factor as defined in Sec. 3.1.3.
r	A characteristic length of the foundation as defined in Sec. 2.5.2.1 (ft or m).
r	Radius of gyration (in. or mm) in Chapter 8.
r_a	The characteristic foundation length defined by Eq. 2.5.2.1.2-2 (ft or m).
r_m	The characteristic foundation length as defined by Eq. 2.5.2.1.2-3 (ft or m).
S	Section modulus based on net cross sectional area of a wall (in.^{3} or mm^3) in Chapter 8.

S_{pr} — Probable strength of precast element connectors (Sec. 6A.5.1).

$\bar{s}_u$ — Average undrained shear strength in top 100 ft (30.5 m); see Sec. 1.4.2, ASTM D2166-91 or ASTM D2850-87.

s_h — Spacing of special lateral reinforcement (in. or mm).

T — The fundamental period (sec) of the building as determined in Sec. 2.3.3.

T — The modal period (sec) of the building modified as appropriate to account for the effective stiffness of the energy dissipation system (Sec. 2A.3.2.1).

$\tilde{T}$, $\tilde{T}_1$ — The effective fundamental period (sec) of the building as determined in Sec. 2.5.2.1.1 and 2.5.3.1.

T_a — The approximate fundamental period (sec) of the building as determined in Sec. 2.3.3.1.

T_p — The fundamental period (sec) of the component and its attachment(s) as defined in Sec. 3.3.3.

T_I — Period of seismic-isolated building (sec) in the direction under consideration as prescribed by Eq. 2.6.3.3.2.

T_m — The modal period of vibration (sec) of the m^{th} mode of the building as determined in Sec. 2.4.5.

T_4 — Net tension in steel cable due to dead load, prestress, live load, and seismic load (Sec. 5.5).

t — Specified wall thickness dimension or least lateral dimension of a column (in. or mm) in Chapter 8.

t_c — Thickness of masonry cover over reinforcing bars measured from the surface of the masonry to the surface of the reinforcing bars (in. or mm) in Chapter 8.

V — The total design lateral force or shear at the base (kip or kN).

V — Shear on a masonry section due to unfactored loads (lb or N) in Chapter 8.

V_b — The total lateral seismic design force or shear on elements of the isolation system or elements below the isolation system as prescribed by Eq. 2.6.3.4.1.

V_c	Portion of nominal shear strength provided by the concrete in reinforced concrete or composite columns (see Ref. 7.2).
V_m	Shear strength provided by masonry (lb or N) in Chapter 8.
V_n	Nominal shear strength (lb or N) in Chapter 8.
V_s	The total lateral seismic design force or shear on elements above the isolation system as prescribed by Eq. 2.6.3.4.2.
V_s	Shear yield strength of steel plate in composite shear wall (Eq. 7.4.7.1.1).
V_s	Shear strength provided by shear reinforcement (lb or N) in Chapters 6 and 8.
V_t	The design value of the seismic base shear as determined in Sec. 2.4.8 (kip or N).
V_u	Required shear strength (lb or N) due to factored loads in Chapters 6 and 8.
V_x	The seismic design shear in Story x as determined in Sec. 2.3.5 or Sec. 2.4.8 (kip or kN).
$\tilde{V}_1$	The portion of the seismic base shear, $\tilde{V}$, contributed by the fundamental mode, Sec. 2.5.3 (kip or kN).
ΔV	The reduction in V as determined in Sec. 2.5.2 (kip or kN).
ΔV_1	The reduction in V_1 as determined in Sec. 2.5.3 (kip or kN).
v_s	The average shear wave velocity for the soils beneath the foundation at large strain levels, Sec. 2.5.2 (ft/s or m/s).
$\overline{v}_s$	Average shear wave velocity in top 100 ft (30 m); see Sec. 1.4.2.
v_{so}	The average shear wave velocity for the soils beneath the foundation at small strain levels, Sec. 2.5.2 (ft/s or m/s).
W	The total gravity load of the building as defined in Sec. 2.3.2 (kip or kN).
W	For calculation of seismic-isolated-building period, W is the total seismic dead load weight of the building above the isolation interface.

$\bar{W}$ — The effective gravity load of the building as defined in Sec. 2.5.2 and 2.5.3 (kip or kN).

W_D — The energy dissipated per cycle at the story displacement for the design earthquake (Sec. 2A.3.2).

$\bar{W}_m$ — The effective modal gravity load determined in accordance with Eq. 2.4.5-1 (kip or kN).

W_p — Component operating weight (lb or N).

w — Width of a wood shear panel or diaphragm in Chapter 9 (ft or mm).

w — Moisture content (in percent), ASTM D2216-92.

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

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

x — Elevation in structure of a component addressed by Chapter 3.

Y_{con} — Distance from top of steel beam to top of concrete slab or encasement (Eq. 7.5.2.2).

y — Elevation difference between points of attachment in Chapter 3.

y_p — The distance, in feet (mm), between the center of rigidity of the isolation system rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration.

α — The relative weight density of the structure and the soil as determined in Sec. 2.5.2.1.

α — Angle between diagonal reinforcement and longitudinal axis of the member (degree or rad).

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. 2.5.2.1.

β_o — The foundation damping factor as specified in Sec. 2.5.2.1.

β_I — Effective damping of the isolation system as prescribed by Eq. 2.6.9.5.

γ	Lightweight concrete factor (Sec. 6.2.4.1).
γ	The average unit weight of soil (lb/ft^3 or kg/m^3).
Δ	The design story drift as determined in Sec. 2.3.7.1 (in. or mm).
Δ	The displacement of the dissipation device and device supports across the story (Sec. 2A.3.2.1).
Δ	Suspended ceiling lateral deflection (calculated) in Sec. 3.2.6.4.2 (in. or mm).
Δ_a	The allowable story drift as specified in Sec. 2.2.7 (in. or mm).
Δ_m	The design modal story drift determined in Sec. 2.4.6 (in. or mm)
Δ_p	Relative displacement that the component must be designed to accommodate as defined in Sec. 3.2.2.2 or 3.3.2.2.
δ_{cr}	Deflection based on cracked section properties (in. or mm) in Chapter 8.
Δ_I^+	Maximum positive displacement of an isolator unit during each cycle of prototype testing.
Δ_I^-	Maximum negative displacement of an isolator unit during each cycle of prototype testing.
δ_{max}	The maximum displacement at Level x (in. or mm).
δ_{avg}	The average of the displacements at the extreme points of the structure at Level x (in. or mm).
δ_x	The deflection of Level x at the center of the mass at and above Level x, Eq. 2.3.7.1 (in. or mm).
δ_{xe}	The deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Sec. 2.3.7.1 (in. or mm).
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. 2.4.6 (in. or mm).
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. 2.4.6-3 and 2.5.3.2-1 (in. or mm).

δ_x, δ_{x1}	The deflection of Level x at the center of the mass at and above Level x, Eq. 2.5.2.3 and 2.5.3.2-1 (in. or mm).
ϵ_{mu}	Maximum usable compressive strain of masonry (in./in. or mm/mm) in Chapter 8.
θ	The stability coefficient for P-delta effects as determined in Sec. 2.3.6.2.
τ	The overturning moment reduction factor (Sec. 2.3.6).
ρ	Ratio of the area of reinforcement to the net cross sectional area of masonry in a plane perpendicular to the reinforcement in Chapter 8.
ρ_b	Reinforcement ratio producing balanced strain conditions in Chapter 8.
ρ_h	Ratio of the area of shear reinforcement to the cross sectional area of masonry in a plane perpendicular to the reinforcement in Chapter 8.
ρ_s	Spiral reinforcement ratio for precast prestressed piles in Sec. 4.5.3.4.
ρ_v	Ratio of vertical or horizontal reinforcement in walls (Ref. 7-2).
ϕ	The capacity reduction factor.
ϕ	Resistance factor for steel in Chapters 5 and 7.
ϕ	Strength reduction factor in Chapters 6 and 8.
ϕ_c	Resistance factor for composite columns (= 0.85) in Chapter 7.
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. 2.4.5.
ρ_v	Ratio of vertical or horizontal reinforcement in walls (see Chap. 21 of Ref. 7.2).
Ω	Factor of safety in Chapter 5.

Appendix A

DIFFERENCES BETWEEN THE 1991 AND 1994 EDITIONS OF THE *NEHRP RECOMMENDED PROVISIONS*

ORGANIZATIONAL AND EDITORIAL CHANGES

The structure of the *Provisions* has been revised for the 1994 Edition:

- The 1991 Chapter 2 definitions and symbols have been moved to "Glossary" and "Notations" sections at the conclusion of the *Provisions* volume.
- 1991 Chapter 3, "Structural Design Requirements"; Chapter 4, "Equivalent Lateral Force Procedure"; Chapter 5, "Modal Analysis Procedure"; and Chapter 6, "Soil-Structure Interaction Effects" have been combined into one chapter for the 1994 Edition -- Chapter 2, "Structural Design Criteria, Analysis, and Procedures." New sections with provisions concerning nonbuilding structures and seismically isolated structures have been incorporated into this chapter as has an appendix on passive energy dissipation systems.
- 1991 Chapter 7 ("Foundation Design Requirements") has become 1994 Chapter 4.
- 1991 Chapter 8 ("Architectural, Mechanical, and Electrical Components and Systems") has become 1994 Chapter 3.
- 1991 Chapter 9 ("Wood") remains Chapter 9 in the 1994 Edition.
- 1991 Chapter 10 ("Steel"), has become 1994 Chapter 5.
- 1994 Chapter 11 ("Reinforced Concrete") has become 1994 Chapter 6 and a new appendix on precast concrete elements interconnected using dry connections has been added.
- 1991 Chapter 12 ("Masonry") has become 1994 Chapter 8. Note that the 1991 "Appendix to Chapter 8" has become the main body of the chapter and the 1991 chapter text has been moved to an appendix for the 1994 Edition.
- A new Chapter 7, "Composite Steel and Concrete Structure Design Requirements," has been added for the 1994 Edition.
- Figures and equations have been relabeled in the 1994 Edition with the section numbers in which they appear (tables were so numbered beginning in the 1991 Edition).

To facilitate use of the 1994 Edition by those familiar with the 1991 Edition section numbers, this appendix concludes with a comparison of the contents of the two editions.

In addition, several subjects discussed in the 1991 "Commentary to Chapter 1" have been moved to appendixes in the 1994 *Commentary* volume to facilitate use of the commentary material.

Finally, a first step in adding metric units has been taken for the 1994 Edition. The reader is cautioned, however, to use considerable care in applying metric units in design since differences exist among the various disciplines with respect, for example, to whether conversions to metric units are soft or hard, the degree of rounding applied in conversion, and whether metric units have been adopted for specific industry products.

CHAPTER 1, GENERAL PROVISIONS

Provisions and *Commentary* Sec. 1.1 have been modified to provide specific information in the first paragraphs of the *Provisions* and *Commentary* describing the input parameters in the seismic design and construction process and the variabilities associated with the input parameters. The objective of the proposed change is to emphasize the variabilities in the design and construction process in order to make the engineer more aware of the importance of judgments made in interpreting and adapting the basic principles embodied in the *Provisions*.

Provisions Sec. 1.2 and 1.3.1 have been revised to group the exceptions concerning certain wood dwellings and have been revised as indicated below to reflect new soil coefficients introduced in the 1994 Edition.

The text in 1991 Sec. 1.4.1.2 requiring the regulatory agency to indicate its determination of A_a and A_v as if the *Provisions* were to be directly adopted has been deleted since, as indicated in the "Chapter 1 Commentary," the *Provisions* is a source document and is not intended to be an enforceable legal document.

As a result of studies conducted by the BSSC 1994 Design Values Panel and work expected to be performed in updating the *Provisions* for issuance in 1997, a cautionary statement about seismic exposure in some regions has been added to *Commentary* Sec. 1.4.1 and a reference to this section has been added to Maps 1 through 4. In addition, the 1991 Edition "Appendix to Chapter 1" has been replaced with an informative description of the work under way to develop consensus design values maps. Maps 5 through 12, with some revisions (especially in the Pacific Northwest) are included as are two maps developed by the BSSC Design Values Panel during the effort to prepare the 1994 *Provisions* and a draft map of recommendations developed for the state of New York.

Sec. 1.4.2 introduces new seismic coefficients, C_a and C_v, and new soil profile definitions. The soil profile definitions are based on recommendations generated at a Site Response Workshop sponsored by the BSSC, the National Center for Earthquake Engineering Research, and the Structural Engineers Association of California. Briefly, the Soil Profile Type definitions are (see Sec. 1.4.2 for full definitions):

A = Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/sec (1500 m/s).

B = Rock with 2,500 ft/sec $< \bar{v}_s \leq 5{,}000$ ft/sec (760 m/sec $< \bar{v}_s \leq 1500$ m/s).

C = Very dense soil and soft rock with 1,200 ft/sec $< \bar{v}_s \leq 2{,}500$ ft/sec (360 m/s $< \bar{v}_s \leq$ 760 m/s).

D = Stiff soil with 600 ft/sec $\leq \bar{v}_s \leq$ 1,200 ft/sec (180 m/s $\leq \bar{v}_s \leq$ 360 m/s).

E = A soil profile with $\bar{v}_s <$ 600 ft/sec (180 m/s) or any profile with more than 10 ft (3 m) of soft clay defined as soil with PI > 20, $w \geq$ 40 percent, and $s_u <$ 500 psf (25 kPa).

F = Soils requiring site-specific evaluations.

Site coefficients, F_a and F_v, associated with the new soil profile definitions vary with acceleration. For new Soil Profile Type A, F_a and F_v are equal to 0.8 and for Soil Profile B, they are equal to 1.0 in all cases. For Soil Profile Types C, D, and E, F_a and F_v vary with acceleration. In the 1991 Edition, the largest site coefficient was S_4 = 2.0 whereas the largest site coefficient in the 1994 Edition is 3.5.

The site coefficients are incorporated into design through the use of two new seismic coefficients--C_a, which equals $F_a A_a$, and C_v, which equals $F_v A_v$. Throughout the 1994 Edition of the *Provisions*, C_a and C_v replace the 1991 Edition products $A_a S$ and $A_v S$, respectively. In addition, in the 1994 Edition, C_a replaces A_a and A_v in some, but not all, situations. Most of the changes result in an increase in design force; however, for Soil Profile Type B sites, the changes will have no effect on design and for Soil Profile Type A sites, the design forces will decrease.

Only one significant non-force control factor has been changed in the 1994 Edition--i.e., the Sec. 1.2 exemptions for one- and two-family dwellings have been linked to C_a (in the 1991 Edition they were dependent on A_v alone). In regions of higher seismicity ($A_v \geq$ 0.15), the change has no practical effect; however, in regions of lower seismicity where previously all one- and two-family dwellings were exempt, some dwellings will be exempt and other will not, depending on the Soil Profile of the site.

In Sec. 1.4.4 of the 1994 Edition, the Seismic Performance Category for Seismic Hazard Exposure Group III buildings with values of A_v ranging from 0.15 to 0.20g was increased from C to D to reduce the risk of collapse in essential services buildings in regions of moderate seismicity.

In Sec. 1.6 of the 1994 Edition, Seismic Performance Category D is added to Item 2 so that the quality assurance provisions apply to all designated seismic systems in zones with C_a equal to or greater than 0.20 (UBC Zones 3 and 4). All other sections on quality assurance have been revised to meet current standards and other changes in the *Provisions*.

CHAPTER 2, STRUCTURAL DESIGN CRITERIA, ANALYSIS, AND PROCEDURES (Chapters 3, 4, 5, and 6 in the 1991 Edition)

Table 2.2.2, "Structural Systems" (Table 3.3 in the 1991 Edition), has been revised as follows:

- The term "unreinforced masonry shear walls" has been changed to "plain (unreinforced) masonry shear walls" to reflect the terminology in Chapter 8.

- Under bearing wall systems, "Plain concrete shear walls" with an R value of 1-1/2 and a C_d value of 1-1/2 has been added. Height limits are not limited for Seismic Performance

Categories A and B or for Category C when nominal reinforcement per Sec. 8.3.7.2 is provided; the system is not permitted for Categories D and E.

- Under building frame systems, "Plain concrete shear walls" with an R value of 2 and a C_d value of 2 has been added. Height limits are not limited for Seismic Performance Categories A and B or for Category C when nominal reinforcement per Sec. 8.3.7.2 is provided; the system is not permitted for Categories D and E.

- The R value for ordinary moment frames of reinforced concrete has been increased from 2 to 3 and the C_d value from 2 to 2-1/2.

- The R value for intermediate moment frames of reinforced concrete has been increased from 4 to 5 and the C_d value from 3-1/2 to 4-1/2.

- For concentrically braced frames in the building frame system category in the column for Category E, the reference to footnote "g" has been eliminated and replaced with 100.

- Under building frame systems, "Special concentrically braced frame of steel" with an R value of 6 and C_d value of 5 has been added. There are no height limitations for Categories A, B, and C but height is limited to 160 for Category D and to 100 for Category E.

- Under dual system with OMF, "Special concentrically braced frame of steel" with an R value of 6 and C_d value of 5 has been added. There are no height limitations for Categories A, B, and C but height is limited to 160 for Category D and to 100 for Category E.

- Under the dual system with SMF, "Special concentrically braced frames of steel" with an R value of 8 and C_d value of 6-1/2 has been added. There are not height limitations for any Seismic Performance Category.

- Values are added for the composite concrete and steel systems presented in the new Chapter 7.

Provisions for seismically isolated structures have been added to the 1994 Edition as Sec. 2.6. These provisions are based on Appendix Chapter 23, Division III, of the 1994 *Uniform Building Code (UBC)*. This *UBC* appendix has been modified, as required, to conform to the strength-design approach and nomenclature of the *NEHRP Recommended Provisions*.

Provisions concerning nonbuilding structures also have been added to the 1994 Edition as Sec. 2.7. The nonbuilding structures covered are those self-supporting structures that are not buildings but that do require seismic design and that are within the purview of the building official. Examples of nonbuilding structures that are generally the concern of building officials include nonbuilding structures with structural systems similar to those for buildings like pipe rack structures, tanks, vessels, silos, chimneys, stacks, trussed towers, cooling towers, bins and hoppers, storage racks, signs and billboards, amusement structures, and monuments. (Note that certain structures including nuclear power plants, dams, and highway bridges are not covered by the *Provisions* because of their uniqueness.)

A new "Appendix to Chapter 2" presents provisions covering passive energy dissipation systems. This appendix is based on *Tentative Seismic Design Requirements for Passive Energy Dissipation Systems* developed by the Energy Dissipation Working Group of the Base Isolation Subcommittee of the Seismology Committee of the Structural Engineers Association of Northern California; technical information published by the Applied Technology Council in *Proceedings of the ATC 17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control* (March 1993); and papers collected by the Earthquake Engineering Research Institute in a special issue of *Earthquake SPECTRA* (Vol. 8, No. 3, August 1993).

CHANGES IN CHAPTER 3, ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS (Chapter 8 in the 1991 Edition)

The requirements for architectural, mechanical, and electrical components have been completely revised. Each component is assigned a component importance factor, I_p, and new force and displacement equations based on actual data and performance during strong earthquake motions are provided. A more rational basis for connection design is provided and relative displacements are covered specifically. Reference also is made to industry design, fabrication, and installations standards with seismic requirements deemed to meet those of the *Provisions*.

The revised chapter includes requirements for interior nonstructural walls and partitions; cantilever elements; exterior nonbearing walls and panels; cladding materials; suspended ceilings; access floors; storage racks and cabinets; boilers, furnaces, pressure vessels, and related equipment; piping systems; storage tanks; HVAC systems; elevator components; towers; general electrical equipment; and lighting fixtures.

CHANGES IN CHAPTER 4, FOUNDATION DESIGN REQUIREMENTS (Chapter 7 in the 1991 Edition)

Sec. 4.5.2 and 4.5.3 (Sec. 7.5.2 and 7.5.3 in the 1991 Edition) have been modified to conform to the new soil profile definitions.

CHANGES IN CHAPTER 5, STEEL STRUCTURE DESIGN REQUIREMENTS (Chapter 10 in the 1991 Edition)

The new *Seismic Provisions for Structural Steel Buildings*, AISC, June 1992, has been adopted for the 1994 Edition as Ref. 5-3 and, consequently, this chapter of the *Provisions* has been greatly shortened. Ref. 5-3 is divided into two parts: Part I is based on the AISC *Load and Resistance Factor Design Specification* (LRFD), Ref. 5-1. Part II is based on the AISC *Allowable Stress Design Specification* (ASD), Ref. 5-2, but the allowable stresses are multiplied by 1.7 to convert to nominal strengths. Simplified ϕ factors are given in Part II to allow the user to develop design strengths that can be used with the limit state seismic loads of the *Provisions*.

Note that based on the unexpected structural damage to steel moment frames from the 1994 Northridge earthquake and pending the results of ongoing and future research studies, it was concluded that use of the welded flange and welded or bolted web special moment frame beam to column connection in Sec. 8.2.c of Ref. 5-3 should be supplemented by the addition of steel flange plates or equivalent measures. Accordingly, the prescriptive Sec. 8.2.c has been suspended and replaced with a performance requirement intended to achieve ductile yielding of the special moment frame under severe cyclic loads. The expected strength ratios of the beam and column materials should be considered, and good welding practice must be followed.

The provisions concerning concentrically braced frames have been updated for the 1994 Edition, and detailing requirements for special concentrically braced frames have been added.

CHANGES IN CHAPTER 6, CONCRETE STRUCTURE DESIGN REQUIREMENTS (Chapter 11 in the 1991 Edition)

This chapter has been revised to be applicable to plain as well as to reinforced concrete. It also has been updated to provide design provisions for precast systems emulating monolithic reinforced concrete construction. A new appendix has been added to introduce provisions for structural systems composed of precast concrete elements interconnected using dry connections. Further, the chapter has been revised to reflect the 1992 reformatting of Chapter 21 of *Building Code Requirements for Reinforced Concrete*, American Concrete Institute, ACI 318, and anchor bolt requirements have been changed.

NEW CHAPTER 7, COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS

This new *Provisions* chapter presents design and detailing requirements for composite structures that are expected to provide structural toughness, ductility, strength, and stiffness equivalent to comparable concrete and steel structures. The corresponding *Commentary* chapter describes composite systems and presents considerable additional information regarding design requirements.

CHAPTER 8, MASONRY STRUCTURE DESIGN REQUIREMENTS (Chapter 12 in the 1991 Edition)

This chapter has been changed significantly for the 1994 Edition in that the limit states design provisions introduced in an appendix in the 1991 Edition have been adopted into the main body of the *Provisions* while the allowable stress provisions have been moved to an appendix as an alternate set of design requirements. New strength reduction factors are introduced and design methods are described.

CHANGES IN CHAPTER 9, WOOD STRUCTURE DESIGN REQUIREMENTS

The provisions for wood structures have been generally revised and updated in light of research and experience data gathered since the 1991 Edition. Significant changes have been made for conventional construction in light of the effects of the 1994 Northridge earthquake.

COMPARISON of the CONTENTS of the 1991 AND 1994 *PROVISIONS*

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