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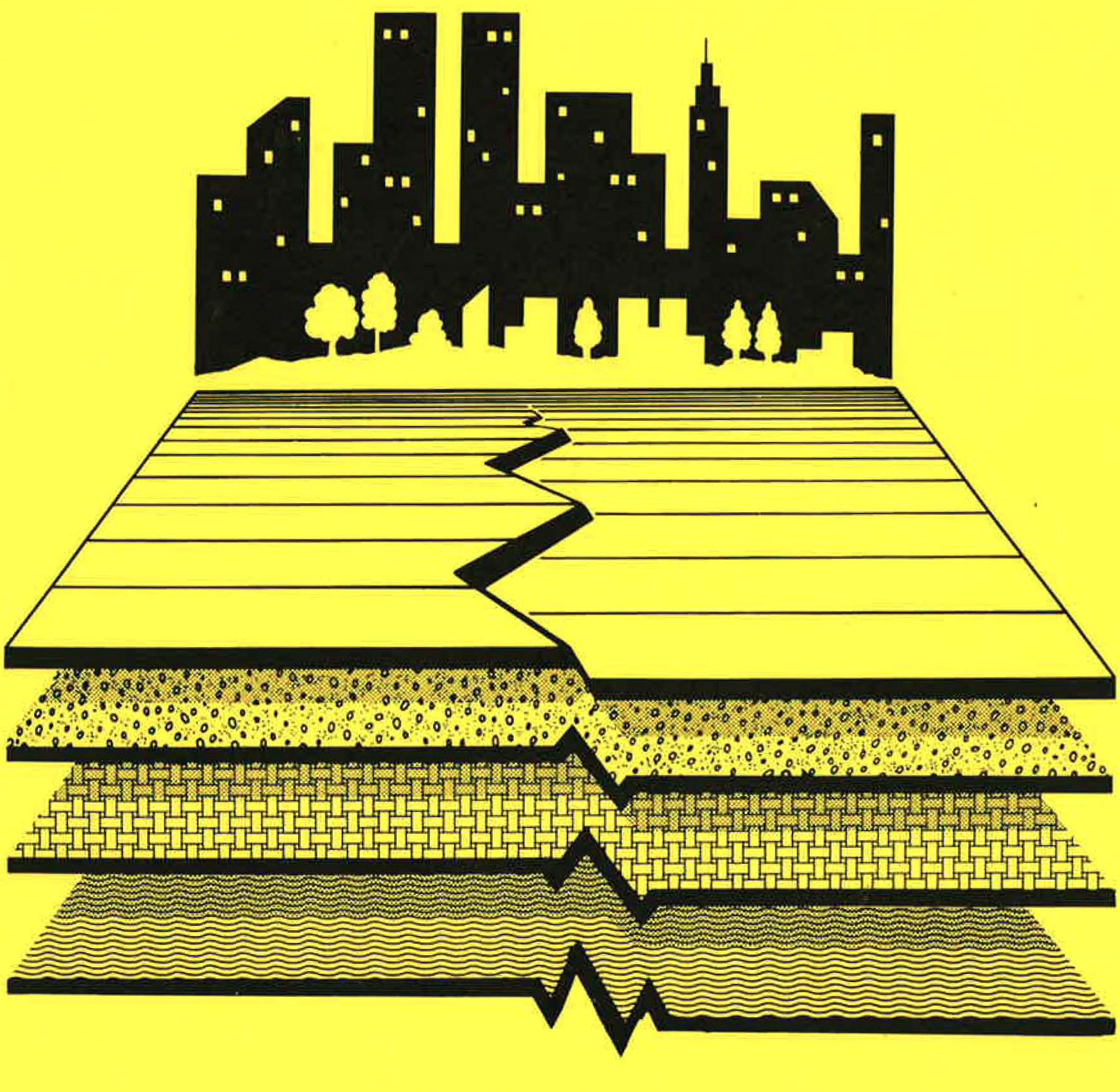
1985 EDITION

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

Part 1
Provisions



EARTHQUAKE HAZARDS REDUCTION SERIES 17

BSSC PROGRAM ON IMPROVED SEISMIC SAFETY PROVISIONS

1985 EDITION

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

Part 1
Provisions

BUILDING
SEISMIC
SAFETY
COUNCIL

BUILDING SEISMIC SAFETY COUNCIL

The Building Seismic Safety Council (BSSC) is an independent, voluntary body that was established under the auspices of the National Institute of Building Sciences (NIBS) in 1979 as a direct result of nationwide interest in the seismic safety of buildings. It has a membership of 57 organizations 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:

1. Promotes the development of seismic safety provisions suitable for use throughout the United States;

2. Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;

3. Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;

4. Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;

5. 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;

6. Advises government bodies on their programs of research, development, and implementation; and

7. Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building-type structures and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations.

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 strutured to provide all interested entities (e.g., 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 standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

BSSC PROGRAM ON IMPROVED SEISMIC SAFETY PROVISIONS

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

1985 Edition

PART 1

PROVISIONS

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

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

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
1985

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

For further information regarding this document, contact the Executive Director, Building Seismic Safety Council, 1015 15th Street, N.W., Suite 700, Washington, D.C. 20005.

Reports prepared by the Building Seismic Safety Council as part of its Program on Improved Seismic Safety Provisions include the following:

Plan for Stimulating Maximum Utilization of the NEHRP Recommended Provisions, 1986
Improving the Seismic Safety of New Buildings: A Nontechnical Explanation of the NEHRP Recommended Provisions, 1986
Handbook for Earthquake Resistant Building Design: Application of the 1985 Edition of the NEHRP Recommended Provisions, 1986
Guidelines for Preparing Code Changes Based on the NEHRP Recommended Provisions, 1986
The Potential Impact on the Building Regulatory System of Using the NEHRP Recommended Provisions, 1986
NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1985 Edition, 3 Parts, 1985
Societal Implications: A Community Handbook, 1985
Societal Implications: Selected Readings, 1985
Overview of Phases I and II of the BSSC Program on Improved Seismic Safety Provisions, 1984
Trial Designs, 17 Volumes, 1984

The following materials on existing buildings were developed for FEMA during the ABE Joint Venture (conducted by the Applied Technology Council, Building Seismic Safety Council, and Earthquake Engineering Research Institute) and are available from FEMA, Earthquake Programs, Washington, D.C. 20472:

Proceedings: Workshop on Reducing Seismic Hazards of Existing Buildings, 1985
An Action Plan for Reducing Earthquake Hazards of Existing Buildings, 1985

Printed in the United States of America.

PREFACE

The Federal Emergency Management Agency (FEMA) is pleased to have the opportunity to sponsor the Program on Improved Seismic Safety Provisions being conducted by the Building Seismic Safety Council (BSSC). The materials produced by this program represent the first new tangible results of a significant effort, under way for several years, to lessen the seismic effects on buildings. This 1985 edition of the *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings* is particularly worthy of attention. It was balloted twice by the BSSC member organizations and received very strong support. We look forward to the widespread use of the 1985 *NEHRP Recommended Provisions* by design and regulatory entities in developing stronger seismic regulations for new buildings. One of the main objectives of the *Earthquake Hazards Reduction Act of 1977* is thereby being met.

This significant seismic mitigation effort is the product of a broad consensus of representatives of the American building industry. It was made possible through very generous contributions of time and expertise on the part of many individuals over many years. FEMA compliments the participants in this program and gratefully acknowledges their efforts. Particular thanks are due to Dr. James R. Harris, J. R. Harris and Company, and Dr. Edgar V. Leyendecker, Center for Building Technology, National Bureau of Standards, respectively the Co-chairman and Secretariat of the BSSC Technical Overview Committee, and to Mr. William W. Moore, Dames and Moore, and Mr. Roy G. Johnston, Brandow and Johnston, past Chairmen of the BSSC Board of Direction, for their unstinting efforts in behalf of this program.

Federal Emergency Management Agency

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INTRODUCTION

The 1985 edition of the *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions* represents one product of the Building Seismic Safety Council's multiyear, multitask Program on Improved Seismic Safety Provisions, which is described in some detail below. It has the consensus approval of the Council membership (see inside back cover) and is intended to serve as a source document for use by any interested member of the building community.

The provisions and accompanying commentary are based on a document issued by the Applied Technology Council (ATC) in 1978, *Tentative Provisions for the Development of Seismic Regulations for Buildings.*[1] The considerable amount of research conducted since that document was issued generally is not reflected here. In addition, the BSSC itself has identified specific areas in need of further attention and is, in fact, pursuing such matters so that the next edition of the *NEHRP Recommended Provisions*, which should be issued in 1988, will be more definitive. Nevertheless, it is believed that the provisions and commentary provided here represent the best state-of-the-art set of guidelines now available concerning the seismic-resistant design of new buildings.

BACKGROUND OF THE BSSC PROGRAM

The genesis of the effort of which the Building Seismic Safety Council (BSSC) Program on Improved Seismic Safety Provisions is a major element began with initiatives taken by the National Science Foundation (NSF) as a part of its earthquake research support program. Through the National Bureau of Standards (NBS), *Tentative Provisions for the Development of Seismic Regulations for Buildings* (referred to in this report as the *Tentative Provisions*) was prepared by the Applied Technology Council (ATC) and published as ATC 3-06. As the ATC noted, the document included many innovations and a careful assessment was needed:

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[1]The provisions and commentary presented as Parts 1 and 2 of this report forcus on new buildings. Part 3 presents the material concerning existing buildings that was included as Chapters 13 through 15 of the original ATC document; these provisions were not tested or evaluated during the BSSC program and are included solely for the information of the reader. More recent work concerning existing buildings has been sponsored by FEMA and is referenced in Chapter 1, Part 1, of the *NEHRP Recommend Provisions*.

> Because of the many new concepts and procedures included in these tentative provisions, they should not be considered for code adoption until their workability, practicability, enforceability, and impact on costs are evaluated by producing and comparing building designs for the various design categories included in this document.

Following the issuance of the *Tentative Provisions* in 1978, NBS released Technical Note 1100, "Analysis of Tentative Seismic Design Provisions for Buildings." In this note, NBS reported its "...systematic analysis of the logic and internal consistency of the [*Tentative Provisions*]." Based on its determination of the need to deal with unresolved public comments on the *Tentative Provisions* and issues raised in its own analysis, NBS issued a *Plan for the Assessment and Implementation of Seismic Design Provisions for Buildings* in November 1978 as its final submission to NSF. This plan included four tasks:

1. A thorough review of the *Tentative Provisions* by all interested organizations;

2. The conduct of trial designs to establish the technical validity of the new provisions and to predict their economic impact;

3. The establishment of a mechanism to encourage consideration and adoption of the new provisions by organizations promulgating the appropriate national standards and model codes; and

4. Educational, technical, and administrative assistance to facilitate implementation and enforcement.

During this same period, other events significant for this effort were taking place. In October 1977, Congress passed the Earthquake Hazards Reduction Act of 1977 (P.L. 95-124) and the National Earthquake Hazards Reduction Program was released on June 22, 1978. The concept of an independent agency to coordinate all emergency management functions at the federal level also was under discussion. When this concept was effected and the Federal Emergency Management Agency (FEMA) was created, FEMA became the implementing agency with NSF retaining its research-support role.

The emergence of FEMA as the agency responsible for implementation of P.L. 95-124 and the President's program on earthquake hazard mitigation also required establishment of a mechanism for obtaining a broad public and private consensus on both recommended improved housing and building design and construction regulatory provisions and the means to be used in their promulgation. A series of meetings was held between representatives of the original participants in the NSF-sponsored project on seismic design provisions (the ATC; its parent organization, the Structural Engineers Association of California; NSF; and NBS) and FEMA, the American Society of Civil Engineers, and the National Institute of Building Sciences (NIBS). From these meetings, the concept of the Building Seismic Safety Council was born. As the concept began to take form, progressively wider public and private participation was sought, culminating in a broadly representative organizing meeting in the spring of 1979 in St.

Louis, Missouri, at which time the BSSC was established under the auspices of NIBS and a charter and organizational rules and procedures were thoroughly debated and agreed upon.

In essence, the BSSC provided the mechanism envisioned in Task 3 of the 1978 NBS plan (a forum for elements of federal, state, and local governments; standards and model code bodies; professional societies; building industry and trade organizations; the research community; and consumer groups to participate equally in the development of a consensus on improved building seismic safety provisions and the means of their promulgation through the existing public and private building regulatory systems throughout the nation). A joint BSSC-NBS committee then was formed to conduct the review called for in Task 1 of the plan. The review effort concluded in 1980 and resulted in 198 recommendations for changes in the *Tentative Provisions* (*Review and Refinement of ATC 3-06 Tentative Seismic Provisions*, NBSIR 80-2111-11). FEMA provided funds to both the BSSC and NBS to support this activity.

As the review effort drew to a close, the BSSC and NBS created another joint committee (Committee 10A) to develop criteria by which the trial designs called for in Task 2 of the 1978 NBS plan could be evaluated and to recommend a specific trial design program plan. Subsequently, the BSSC created a special BSSC-NBS Trial Design Overview Committee (designated Committee 12 and sometimes referred to as the Technical Overview Committee) and charged it to, among other activities, revise the Committee 10A plan to accommodate a multiphased effort and to refine the *Tentative Provisions*, to the extent practicable, to reflect the recommendations generated during the earler review. (NBS provides the Secretariat for the Overview Committee).

The Overview Committee completed the revised plan in August 1982. It was released in November 1982 as *Plan for a Trial Design Program to Assess Amended ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings* (NBSIR 82-2589/BSSC 82-1). (It should be noted that this plan was modified in some ways during the course of the BSSC program.) The result of the committee's revision effort was released in December 1982 as *Amendments to ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings for Use in Trial Designs* (NBSIR 822626/BSSC 82-2). (The term "amended *Tentative Provisions*" is used in this report to refer to this document in conjunction with the original ATC document.)

It is all of these efforts that provided the basis for the BSSC Program on Improved Seismic Safety Provisions. The basic objectives of the BSSC program are to:

1. Estimate the economic impact that various sectors of the building community would experience in changing from current practice to the amended *Tentative Provisions* with particular attention to design, construction, and regulatory time and cost.

2. Evaluate the usability of the amended *Tentative Provisions* insofar as designers, builders, regulatory officials are concerned including consideration of relevance (i.e., determine whether there is a

reasonable balance between the effort and/or cost involved in carrying out the amended *Tentative Provisions* and the significance of the results), clarity (i.e., determine whether the amended *Tentative Provisions* can be interpreted and applied consistently), completeness (i.e., determine whether the amended *Tentative Provisions* address all the applicable types of buildings and components), and practicability (i.e., determine whether the design procedures required by the amended *Tentative Provisions* are viable and the resulting designs practical to construct).

3. Establish the technical validity of the amended *Tentative Provisions* (i.e., determine whether seismic safety will be enhanced by their adoption).

4. Generate objective information for use in the future resolution of disputes concerning specific provisions.

5. Generate objective information that will be transferable to components, building types, and locations not specifically investigated.

CONDUCT OF THE BSSC PROGRAM

During Phases I and II of the BSSC program, 17 professional design organizations from 9 cities were retained to prepare trial designs of the following building types:

1. Low-, mid-, and high-rise residential buildings (Type R),

2. Mid- and high-rise office buildings (Type O),

3. One-story industrial buildings (Type I), and

4. Two-story commercial buildings (Type C).

Each building was designed twice: once according to the amended *Tentative Provisions* and once according to the prevailing local code for the particular location of the design. Basic structural designs, partial structural designs (special studies to test specific parameters, provisions, or objectives), and partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building) were prepared and design and construction cost estimates were developed. The BSSC-NBS Overview Committee, assisted by a technical consultant, reviewed the design concept and approach at various stages (interpretation of design criteria, analysis of load effects, and completion of design). The design firms were asked to certify the accuracy of their calculations and to report their findings.

During Phase I of the program, 10 design firms were retained to prepare trial designs for 26 new buildings in 4 cities with medium to high seismic risk--10 in Los Angeles, 4 in Seattle, 6 in Memphis, and 6 in Phoenix. During Phase II, 7 firms were retained to prepare trial designs for 20 buildings in 5 cities with medium to low seismic risk--3 in Charleston (S.C.), 4 in Chicago, 3 in Ft. Worth, 7 in New York, and 3 in St. Louis.

The schedule of basic designs and the design firms are presented in Tables 1 and 2, respectively.

Phase II concluded with publication of:

1. A draft version of the recommended provisions, *The NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, that included three parts--the draft provisions, the draft commentary to the provisions, and an appendix that presented Chapters 13-15 concerning existing buildings of the original ATC document;

2. An overview of Phases I and II of the BSSC program that included the BSSC-NBS Overview Committee's analysis of the results and the executive summaries from the reports of the design firms participating in the program as well as a series of appendixes that presented the initial amendments to the original ATC document, the original trial design program plan, the plan for studies to be conducted in Phase III of the program, the detailed contract work plans for Phases I and II, and a list of the members of the BSSC technical committees.

3. The design firms' reports.

The draft provisions issued at the conclusion of Phase II reflected the initial amendments to the original ATC document as well as further refinements made by the Overview Committee during Phases I and II of the program. They represented an interim set of provisions pending their first balloting by the BSSC member organizations, which began in July 1984. This first ballot was conducted on a chapter-by-chapter basis using a ballot that provided for four responses: yes, yes with reservations, no, and abstain. (The "yes with reservations" and "no" votes were to be accompanied by an explanation of the reasons for the vote and the "no" votes were to be accompanied by specific suggestions for change if those changes would change the negative vote to an affirmative).

During Phase III of the program, all comments and "yes with reservation" and "no" votes received as a result of the first ballot were compiled. Proposals for dealing with these responses then were developed for consideration by the Technical Overview Committee and, subsequently, the BSSC Board of Direction. The draft provisions were then revised to reflect the changes deemed appropriate by the BSSC Board and were submitted to the BSSC membership for balloting again in August-September 1985.

As a result of this second ballot, virtually the entire provisions document received consensus approval, and a special BSSC Council meeting was held in November 1985 to resolve the remaining differences. Some aspects of the provisions are being referred to BSSC Technical Committee for further study during the updating effort. These committees also will consider any new developments that come to light so that subsequent editions of the provisions will be as up-to-date as possible.

TABLE 1 Schedule of Basic Trial Designs

Plan Form	Vertical Load System	Seismic Resisting System Components	Other Vertical Components	Floor or Roof Components	Bldg. No.	No. of Stories	Cities: Phase I				Phase II				
							Los Angeles	Seattle	Memphis	Phoenix	New York	Chicago	Ft. Worth	Charleston	St. Louis
Residential	Bearing Walls	Plywood wall		Wood + plywood diaphragm	1	3	•	•							
		Concrete masonry wall		Wood + plywood diaphragm	2	3				•					
		Brick and concrete masonry		Wood + plywood diaphragm	2A	3						•			
		Concrete masonry wall		Prestressed slab	3	5				•			•		
		Brick wall		Reinforced concrete slab	4	5									
				Reinforced concrete slab	5	12	•				•				
				Reinforced concrete slab	5A	10									•
		Brick and concrete masonry wall		Steel joist	6	5								•	
				Steel joist	7	12									
		Reinforced conrete wall		Reinforced concrete slab	8	5			•						
				Reinforced concrete slab	9	12						•			
				Post-tensionsed slab	10	5				•					
		Precast concrete wall		Prestressed slab	11	5									
				Prestressed slab	12	12									
	Complete Vertical Load Carrying Frame	Steel, braced frame (transverse)/ moment frame (longitudinal)	Steel Framing	Steel joists	13	10									
			Steel Framing	Steel beam & RC slab	14	20			•						
		Reinforced concrete shear wall	RC framing	RC flat plate	15	10	•						•		
			RC framing	Post-tensioned flat plate	16	20						•			•
			RC framing	Post-tensioned flat plate	17	30									
		Reinforced concrete moment frame (perimeter)	RC framing	RC flat plate	18	10	•		•						
			RC framing	RC flat plate	19	20									
		RC, MF (perimeter) & SW (dual)	RC framing	RC flat plate	20	20									
			RC framing	RC flat plate	20A	30					•				
Office	Bearing walls	Reinforced concrete wall (core)	RC framing	RC flat slab	21	10									
			RC framing	RC flat slab	22	20				•	•				
		PC wall (interior & exterior)	PS framing	Prestressed slab	23	10									
	Complete Vertical Load Carrying Frame	Reinforced concrete shear wall	Steel framing	Steel beam & RC slab	24	10		•						•	
			Steel framing	Steel beam & RC slab	25	20									
		Steel braced frame	Steel framing	Steel beam & RC slab	26	20									
			Steel framing	Steel beam & RC slab	26A	5									•
		Steel moment frame	Steel framing	Steel beam & RC slab	27	10	•		•						
			Steel framing	Steel joist	M27A	5					•				
			Steel framing	Steel beam & RC slab	P27A	5							•		
			Steel framing	Steel beam & RC slab	28	40									
			Steel framing	Steel beam & RC slab	28A	30					•				
		Steel MF & RC SW (dual)	Steel framing	Steel beam & RC slab	29	20	•								
		Steel MF and BF (dual)	Steel framing	Steel beam and RC slab	30	20		•							
		Reinforced concrete moment frame	RC framing	Post-tensioned flat slab	31	10									
			RC framing	RC pan joist & waffle	32	10				•	•				
			RC framing	PT pan joist	33	20									
		RC, MF & SW (dual)	RC framing	RC pan joist & waffle	34	20	•								
Industrial	Bearing walls	Concrete masonry wall	Steel framing	Steel joist	35	L				•					
		PC wall (maybe PS)	PS framing	Prestressed slab	36	H									
			PS framing	PC double tees girders & beam	36A	H						•			
		PC tilt-up wall	Wood framing	Wood (plywood)	37	L	•								
	Complete Vertical Load Carrying Frame	PC tilt-up wall	Steel framing	Steel joist	38	L			•						
		Steel moment frame (transverse)/ braced frame (longitudinal)	Steel framing	Steel purlins & deck	39	L	•							•	
			Steel framing	Steel long-span truss	40	H		•							
		Concrete masonry wall	Steel framing	Steel joist	41A	2					•				
Commercial	Complete Vertical Load Carrying Frame	Concrete masonry wall	Steel framing	Steel joist	41	2	•								
			Steel framing	Steel joist (irregular plan form)	42	2			•						
		PS moment frame	PS framing	Prestressed slab	43	2									

1 - All office buildings will have a high first story, the industrial buildings are all on one story (with the exception of Building No. 41A) and for them the L indicates a low clearance, and H indicates a high clearance.

2 - BF = braced frame; MF = moment frame; RC = reinforced concrete; PC = precast concrete; PT = post-tensioned concrete; SW = shear wall (non-bearing); PS = prestressed, precast concrete

3 - With the exception of the industrial building with purlins and steel deck (the metal building) all moment frames in Los Angeles, Seattle, and Memphis are to be Special. All moment frame in dual systems must also be Special. All other moment frames may be Ordinary.

TABLE 2 Design Firms and Types of Building Designs

City/Design Firm	Type of Building/Number from Table 1
Charleston, S.C.	
Enwright Associates, Inc.	o 5-Story Brick and RC Block Bearing Walls (R)/CSC-6 o 10-Story Steel Frame with RC Shear Walls (O)/CSC-24 o 1-Story Steel Moment and Braced Frame (I)/CSC-39
Chicago	
Alfred Benesch and Company	o 3-Story Brick and RC Block Bearing Walls with Plywood Floor & Roof Diaphragms (R)/C-2A o 20-Story RC Frame with RC Shear Walls (R)/C-16
Klein and Hoffman, Inc.	o 12-Story RC Bearing Wall (R)/C-9 o Parametric Study of Steel Moment and/or Braced Frames (O)/C-26, C-27, & C-30 o 1-Story Precast RC Bearing Walls with PC Double Tee Roof (I)/C-36A
Ft. Worth, Texas	
The Datum/Moore Partnership	o 5-Story RC Block Walls with Prestressed Slabs (R)/FW-3 o 10-Story RC Frame with RC Shear Walls (R)/FW-15 o 5-Story Steel Moment Frame (O)/FW-27A
Los Angeles	
S. B. Barnes and Associates	o 3-Story Wood with Plywood Walls (R) /LA-1 o 1-Story Wood Frame with Precast Concrete Tilt-Up Walls (I)/LA-37 o 1-Story Steel with Moment and Braced Frames (I)/LA-39 o 2-Story Steel Frame with RC Block Walls (C)/LA-41
Johnson and Nielsen Associates	o 20-Story Steel Moment Frame with Shear Walls (Dual) (O)/LA-34

TABLE 2 Continued

City/Design Firm	Type of Building/Number from Table 1
Wheeler and Gray	o 12-Story Reinforced Brick Bearing Wall with RC Slabs (R)/LA-5 o 10-Story RC Frame with Shear Walls (R)/LA-15 o 10-Story RC Frame (Perimeter) with RC Slabs (R)/LA-18 o 10-Story Steel Moment Frame (O)/LA-27
<u>Memphis</u>	
Allen and Hoshall	o 5-Story Bearing Wall (R)/M-8 o 1-Story Steel Frame with RC Tilt-Up Exterior Shear Walls (I)/M-38 o 2-Story Steel Frame with Non-Bearing RC Block Walls (C)/M-42
Ellers, Oakley, Chester and Rike, Inc.	o 20-Story Steel Moment and Braced Frame with RC Slab Floors (R)/M-14 o 10-Story RC Moment Frame (Perimeter) (R)/M-18 o 10-Story Steel Moment Frame (Special) with RC Slabs (O)/M-27
<u>New York City</u>	
Weidlinger Associates	o 12-Story Brick Bearing Wall (R)/NY-5 o 30-Story RC Moment Frame and Non-Bearing Shear Wall (Dual) (R)/NY20A o 10-Story RC Moment Frame (O)/NY-32
Robertson, Fowler and Associates P.C.	o 20-Story RC Bearing Wall (O)/NY-22 o 5-Story Steel Moment Frame (O)/NY-27A o 30-Story Steel Moment Frame (O)/NY-28A o 2-Story Steel Frame with RC Block Walls (I)/NY-41A
<u>Phoenix</u>	
Magadini-Alagia Associates	o 5-Story RC Bearing Wall (R)/P-10 o 20-Story RC Bearing Wall with Core Shear Walls (O)/P-22 o 10-Story RC Frame (Ordinary) (O)/P-32

TABLE 2 Continued

City/Design Firm	Type of Building/Number from Table 1
Read Jones Christoffersen, Inc.	o 3-Story RC Block Bearing Wall (R)/ P-2 o 5-Story RC Block Bearing Wall (R)/P-3 o 1-Story Steel Frame with RC Block Shear Walls (I)/P-35
St. Louis	
Theiss Engineers, Inc.	o 10-Story Clay Brick Bearing Wall (R)/ SL-5A o 20-Story RC Frame with RC Shear Walls (R)/SL-16 o 5-Story Steel Frame with Braced Frames at Core (O)/SL-26A
Seattle	
ABAM Engineers, Inc.	o 10-Story Steel Frame with RC Shear Wall (O)/S-24
Bruce C. Olsen	o 3-Story Wood with Plywood Walls (R)/ S-1 o 1-Story Long Span Steel, 30′ Clear Height-MF and Braced Frames (I)/S-40
Skilling Ward Rogers Barkshire, Inc.	o 20-Story Steel Frame-Dual Special & Braced Frames (O)/S-30

NOTE: Letters in parenthesis denote building type--R = Residential, O = Office, I = Industrial, and C = Commercial.

RELATED BSSC PROJECTS

Also being prepared during Phase III of the BSSC program to support and complement the *NEHRP Recommended Provisions* are:

1. A technical applications handbook,

2. An explanatory version of the provisions for the lay audience,

3. Guidelines for making code changed based on the *Provisions*.

4. A plan to encourage maximum utilization of the *Provisions*.

These documents are expected to be available in early 1986.

A study of the societal implications of using new or improved seismic design provisions has already been completed. The reports resulting from this study, *Societal Implications: Community Handbook* and *Societal Implications: Selected Readings*, have been published as Numbers 13 and 14, respectively, in FEMA's Earthquake Hazards Reduction Series. A study of the regulatory impact of applications of the provisions also is virtually complete.

Finally, all users of the *NEHRP Recommend Provisions* should be aware that the effort to update the provisions is under way, and the second edition is expected to be available in 1988.

ACKNOWLEDGMENTS

In presenting this 1985 Edition of the *NEHRP Recommended Provisions*, the BSSC wishes to recognize the accomplishments of the many individuals and organizations involved over the years. As Past Chairman of the BSSC Board of Direction Roy G. Johnston noted in an earlier version of these provisions:

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

The BSSC program itself has involved similar contributions of time and effort and will continue to do so in the years to come as newer editions of the *NEHRP Recommended Provisions* are developed and published.

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

- o The members of the BSSC-NBS Trial Design Overview Committee, especially Co-Chairman James R. Harris who gave so freely of his time and expertise;

- o The members of the various BSSC technical committees;

- o E. V. Leyendecker of the National Bureau of Standards Center for Building Technology who served as the secretariat for the Overview Committee;

- o Roland Sharpe who served not only as a member of the Overview Committee but also as the BSSC Technical Consultant during the trial design phase of the BSSC program; and

- o Ugo Morelli, the FEMA Project Officer, whose continuing interest and support and prompt assistance with contractual matters were so essential to program continuity.

Special thanks is extended to the trial design firms, each of which produced reports that reflected competent, professional input far beyond that compensated for through contractual arrangements:

- o ABAM Engineers, Inc.
- o Alfred Benesch and Company
- o Allen and Hoshall
- o Bruce C. Olsen
- o The Datum/Moore Partnership
- o Ellers, Oakley, Chester, and Rike, Inc.
- o Enwright Associates, Inc.
- o Johnson and Nielsen Associates
- o Klein and Hoffman, Inc.
- o Magadini-Alagia Associates
- o Read Jones Christoffersen, Inc.
- o Robertson, Fowler, and Associates P.C.
- o S. B. Barnes and Associates
- o Skilling Ward Rogers Barkshire, Inc.
- o Theiss Engineers, Inc.
- o Weidlinger Associates
- o Wheeler and Gray

Appreciation is also due to:

- o James R. Smith, the BSSC Executive Director, whose administrative talents and experience were crucial to conduct of the program;
- o Elaine Griffin, NIBS Secretary, whose clerical skills were so enthusiastically devoted to the program; and
- o Claret M. Heider, the BSSC's Consulting Technical Writer-Editor, whose wordsmithing skills and report organization abilities have contributed significantly to this and earlier BSSC reports.

At this point I, as Chairman, would like to express my personal gratitude to the members of the BSSC Board of Direction and especially to its past chairmen, William W. Moore and Roy G. Johnston, for assuming responsibility for and maintaining control over the program and to all those who provided advice, counsel, and encouragement during the conduct of the effort or who otherwise participated in the development of the *NEHRP Recommended Provisions*.

Warner Howe
Chairman, BSSC Board of Direction

CONTENTS

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

1985 Edition

Part 1--Provisions

Part 2--Commentary

Part 3—Appendix (Existing Buildings)

A revised reference to "Existing Buildings" TG

Chapter 1

GENERAL PROVISIONS

1.1 PURPOSE

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

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

1.2 SCOPE

and of horizontal and vertical additions to buildings,

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These provisions establish requirements for the design and construction of new buildings to resist the effects of earthquake motions. Part 3 of these provisions addresses requirements for strengthening existing buildings when alterations reducing the seismic force resistance are made or when changes in occupancy occur that would result in the assignment of the building to a higher Seismic Performance Category.[1]

A revised reference to "Existing Buildings" is needed.

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[1] Part 3 presents provisions concerning existing buildings that were developed by the Applied Technology Council (ATC) and published in 1978 as Chapters 13 through 15 of ATC Report 3-06, *Tentative Provisions for the Development of Seismic Regulations for Buildings*. They were not considered during the BSSC program leading to the *NEHRP Recommended Provisions* and are included only as guidance for those interested in existing buildings. It should be noted, however, that a comprehensive plan for mitigating seismic hazards in existing buildings was recently completed for FEMA by the ABE Joint Venture (conducted by the Applied Technology Council, Building Seismic Safety Council, Earthquake Engineering Research Institute) and is being evaluated by the appropriate federal agencies. A workshop was held as part of the plan development effort and the proceedings were published by FEMA in September 1985 (*Proceedings: Workshop on Reducing Seismic Hazards of Existing Buildings*). Copies of both the plan and the proceedings are available from FEMA, Earthquake Programs, Washington, D.C. 20472.

EXCEPTION:

The following buildings need not comply with these provisions:

1. Buildings classified for agricultural use and intended only for incidental human occupancy.

2. One- and two-family dwellings that are located in areas having a Seismicity Index of 1 or 2 in Table 1-B. /.4.)

3.

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

To Commentary

1.3 APPLICATION OF PROVISIONS

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

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

1.3.1 New Buildings

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

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

Additions to

1.3.2 Existing Buildings Alterations and Repairs

The repair or alteration of an existing building subject to these provisions either: (1) shall not reduce the lateral force resistance of

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the building below the requirements of these provisions or (2) shall provide for the seismic forces determined in accordance with these provisions including the modifications permitted by Part 3.[2]

1.3.2.1
1.3.2.7

1.3.3 Change of Use

A building subject to these provisions because of a change of use shall be capable of resisting the seismic forces determined in accordance with these provisions including the modifications permitted by Part 3.[2]

1.3.4 Systematic Abatement of Seismic Hazards in Existing Buildings

For guidance concerning the systematic abatement of seismic hazards in existing buildings, see Part 3 of these provisions.[2]

1.4 SEISMIC PERFORMANCE

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

See Note 2 in Minutes 6/11-12/87

1.4.1 Seismicity Index and Design Ground Motions

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

1. Determine the appropriate Map Area for the building site from ~~Figure 1-1 for A_a and~~ Figure 1-2 for A_v.
   1.4.1A

2. ~~Determine the value of A_a and A_v from Table 1-B for the Map Area found in Step 1.~~

3. Determine the Seismicity Index from Table 1-B for the value of A_v as determined above.
   1.4.1

~~[2]See Footnote 1.~~

Relocate Table 1.4.1 to here

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Either 1-Delete or 2-Put in Commentary or 3-Put in Sec 4.7.1

For reference and use by the Regulatory Agency, contour maps for A_a and A_v are provided as Figures 1-3 and 1-4 to supplement Figures 1-1 and 1-2. Where the acceleration parameters are determined using the contour maps,[3] the Seismicity Index shall be determined from the following table:

A_v	Seismicity Index
$A_v < 0.05$	1
$0.05 \leq A_v < 0.15$	2
$0.15 \leq A_v < 0.20$	3
$0.20 \leq A_v$	4

Alternate Section 1.4.1 for Regulatory Agencies That Have Made a Determination of A_a, A_v, and the Seismicity Index

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

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

1.4.2 Seismic Hazard Exposure Groups

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

Note: correlate commentary to agree with this section

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

---

[3] Figures 1-1, 1-2, 1-3, and 1-4 are enclosed for illustration and information only. The BSSC plans to re-establish and convene its Technical Committee on Seismic Risk Maps for the purpose of ensuring consistency between the two approaches and incorporating any new or additional information and data acquired since the development of these Figures. New maps will be prepared and distributed as soon thereafter as is practicable.

Examples of possible Group III facilities are:

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

include the following

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

Examples of possible Group II facilities are:

• Assembly occupancies with an Occupant load exceeding 1000 in a single room,
• Buildings with occupied floors more than 75 ft above grade

Public assembly for 100 or more persons
Open-air stands for 2,000 or more persons
Day care centers
Schools
Colleges
Retail stores with 5,000 square feet of floor area per floor or more than 35 feet in height
Shopping centers with covered malls, over 30,000 square feet gross area excluding parking
Offices over 4 stories in height or more than 10,000 square feet per floor
Hotels over 4 stories in height
Apartment houses over 4 stories in height
Emergency vehicle garages
Ambulatory health facilities
Hospital facilities other than those in Group III
Wholesale stores over 4 stories in height
Factories over 4 stories in height
Printing plants over 4 stories in height
Hazardous occupancies consisting of flammable or toxic liquids including storage facilities for same

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

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

designed to be

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

1.4.3 Seismic Performance Categories

For the purposes of these provisions, all buildings shall be assigned, based on the Seismicity Index established and the Seismic Hazard Exposure Group designated, to a Seismic Performance Category in accordance with Table 1-A.
1.4.3

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

Relocate table 1.4.3 here

1.4.4 Site Limitation for Seismic Design Performance Category D

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

1.5 ALTERNATE MATERIALS AND METHODS OF CONSTRUCTION

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

TC9-1-3

1.6 QUALITY ASSURANCE

or portions thereof

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

1.6.1 Quality Assurance Plan

See Note 3 TC 9 Minutes 6/11-12/87

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

1. Buildings assigned to Group D category for the Designated Seismic Systems.
2. Buildings assigned to Category C for Structural Seismic Resisting Systems.
3. For all other buildings as determined by the local jurisdiction.

in areas having a Seismicity Index of 2 or 3.

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

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

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

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

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

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

who shall be identified as the Owner's Inspector

1.6.2 Special Inspection

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

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

B. Reinforcing Steel. Special Inspection for reinforcing steel shall be as follows:

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

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

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

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

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

E. Structural Masonry. Continuous Special Inspection required during the placement of all masonry units for buildings assigned to Category D. Continuous Special Inspection required during all grouting operations for masonry that is part of the seismic resisting system in buildings assigned to Categories C and D.

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

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

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

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

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

I. Mechanical and Electrical Components. Periodic Special Inspection required during the installation and anchorage of the following components when designated in Chapter 8 as requiring S or G performance:

1. Equipment using combustible energy sources;

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

3. Machinery, reciprocating and rotating type;

4. Piping distribution systems 3 inches or larger; and

5. Tanks, heat exchangers, and pressure vessels.

1.6.3 Special Testing

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

A. Reinforcing and Prestressing Steel. Special Testing of reinforcing and prestressing steel shall be as follows:

1. Sample at fabricator's plant and test reinforcing steel used in reinforced concrete Special Moment Frames and boundary members of reinforced concrete or reinforced masonry shear walls for limitations

on weldability, elongation and actual-to-specified yield, and ultimate-strength ratios.

EXCEPTION:

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

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

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

C. Structural Masonry. Special Testing of structural masonry shall be as follows:

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

2. When f'_m is to be established by prism tests, at least five representative prisms shall be prepared and tested prior to start of work. During construction, at least one sample prism shall be prepared per day but not less than one sample prism per 5,000 square feet of wall area or less than five such sample prisms for any building during the progress of the work.

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

D. Structural Steel. Special Testing of structural steel shall be as follows:

1. Welded connections for Special Moment Frames shall be tested by nondestructive methods conforming to AWS D1.1-85. All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by other approved equivalent methods.

EXCEPTION:

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

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

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

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

1.6.4 Reporting and Compliance Procedures

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

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

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

1.6.5 Approved Manufacturer's Certification

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

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Relocate to end of 1.4.3

TABLE 1-A 1.4.3 **Seismic Performance Category**

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

Relocate to end of Section 1.4.1

1.4.1

TABLE 1-B Coefficients A_a and A_v and Seismicity Index[a]

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

[a]The Seismicity Index values in this table have been controversial and are of economic concern. The rationale for the number of Seismicity Index values and the closely affected Seismic Performance Categories are in need of careful review. It has been suggested that the number of Seismicity Index values and the Seismic Performance Categories should be increased. The BSSC program for updating these provisions will pay particular attention to this subject.

Chapter 2

DEFINITIONS AND SYMBOLS

2.1 DEFINITIONS

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

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

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

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

Area Separation Partition is any partition installed to provide a required fire separation between portions of buildings.

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

Code Required Component is a component required by the Building Code administered by the Regulatory Agency.

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

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

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

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

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

Designated Seismic Systems are the Seismic Resisting System and those architectural, electrical, and mechanical systems and their components that require special performance characteristics.

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

Effective Peak Acceleration and **Effective Peak Velocity-Related Acceleration** are coefficients for determining the prescribed seismic forces and are given in Sec. 1.4.

Frame

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

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

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

TC 6-5 Eccentrically Braced Frame is a diagonally braced frame in which at least one end of each brace frames into a beam a short distance from a beam column joint or from another diagonal brace. These short beam segments are link beams.

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

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

Ordinary Moment Frame is a Space Frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Ordinary Moment Frames shall conform to Sec. 10.4.1 or Sec. 11.3.

Special Moment Frame is a Space Frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. Special Moment Frames shall conform to Sec.-10.5.1 or Sec. 11.5.

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

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

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

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

Joint, Laterally Confined, is a joint that, in the direction under consideration, has the opposite faces confined by members which are monolithic with the joint and cover 75 percent of the width and depth of the joint.

Loads

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

Gravity Load, W, is as defined in Sec. 4.2.

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

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

EXCEPTION:

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

Owners Inspector

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

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

Resilient Mounting System is a system incorporating helical springs, air cushions, rubber-in-shear mounts, fiber-in-shear mounts, or other comparable approved systems.

Resilient Mounting System, Stable, is a system in which the force displacement ratios are equal in the horizontal and vertical directions.

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

> **Restraining Device, Elastic,** is a fixed restraining device that incorporates an elastic element to reduce the seismic forces transmitted to the structure due to impact from the resilient mounting system.
>
> **Restraining Device, Fixed,** is a nonyielding or rigid type of restraining device.
>
> **Restraining Device, Seismic Activated,** is an interactive restraining device that is activated by earthquake motion.

Roofing Unit is a unit of roofing material weighing more than 1 pound.

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

Seismic Hazard Exposure Group is a classification assigned to a buliding based on its use as defined in Sec. 1.4.

Seismic Performance Category is a classification assigned to a building as defined in Sec. 1.4.

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

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

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

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

the Owner's

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

Owner's

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

Relocate this to be in alphabetical order.

Owner's

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

Owner's **Special Inspector** is 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.

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

Utility or Service Interface is the connection of the building's mechanical and electrical distribution systems to the utility or service company's distribution system.

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

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

Bearing Wall is a wall providing support for vertical loads and may be exterior or interior.

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

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

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

2.2 SYMBOLS

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

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

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

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

A_o The area of the load-carrying foundation.

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

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

a_d The incremental factor related to P-delta effects in Sec. 4.6.2.

a_x The amplification factor at level x related to the variation of the response in the height of the building, Sec. 8.3.2.

C_a Coefficient for upper limit on calculated period; see Table 4-A.

C_c The seismic coefficient for components of buildings as specified in Tables 8-B and 8-C (dimensionless).

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

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

$\tilde{C}_s$ The seismic design coefficient determined in Sec. 6.2.1 and 6.3.1 (dimensionless).

C_{sm} The modal seismic design coefficient determined in Sec. 5.5 (dimensionless).

C_T The framing coefficient in Sec. 4.2.2.

C_{vx} The vertical distribution factor as determined in Sec. 4.2.

D_s The total depth of the stratum in Eq. 6-10.

F_i, F_n, F_x	The portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Sec. 4.3.
f'_m	Ultimate compressive strength of masonry.
F_p	The seismic force acting on a component of a building as determined in Sec. 3.7, 8.2, or 8.3.
F_{xm}	The portion of the seismic base shear, V_m, induced at level x as determined in Sec. 5.5.
f_{yh}	The specified yield stress of the Special Lateral Reinforcement, psi.
G	$\gamma v_s^2/g$ = the average shear modulus for the soils beneath the foundation at large strain levels.
G_o	$\gamma v_{so}^2/g$ = the average shear modulus for the soils beneath the foundation at small strain levels.
g	The acceleration due to gravity.
$\bar{h}$	The effective height of the building as determined in Sec.- 6.2 or 6.3.
h_c	The core dimension of a component measured to the outside of the Special Lateral Reinforcement.
h_i, h_n, h_x	The height above the base level, i, n, or x, respectively.
h_{sx}	The story height below level x = $(h_x - h_{x-1})$.
I_o	The static moment of inertia of the load-carrying foundation, Sec. 6.2.1.
i	The building level referred to by the subscript i; i = 1 designates the first level above the base.
K	The stiffness of the equiment support attachment, Sec. 8.3.2.
K_y	The lateral stiffness of the foundation as defined in Sec. 6.2.
K_θ	The rocking stiffness of the foundation as defined in Sec. 6.2.
k	The distribution exponent given in Sec. 4.3.
$\bar{k}$	The stiffness of the building as determined in Sec. 6.2.
L	The overall length of the building (in feet) at the base in the direction being analyzed.

L_o	The overall length of the side of the foundation in the direction being analyzed, Sec. 6.2.1.
M_f	The foundation overturning design moment as defined in Sec. 4.5.
M_o, M_{ol}	The overturning moment at the foundation-soil interface as determined in Sec. 6.2.3 and 6.3.2.
M_t	The torsional moment resulting from the location of the building masses, Sec. 4.4.
M_{ta}	The accidential torsional moment as determined in Sec. 4.4.
M_x	The building overturning design moment at level x as defined in Sec. 4.5 or Sec. 5.8.
m	A subscript denoting the mode of vibration under consideration; i.e., m = 1 for the fundamental mode.
n	Designates the level that is uppermost in the main portion of the building.
P	The performance criteria factor as given in Table 8-A (dimensionless).
P_n	The algebraic sum of the seismic forces and the minimum gravity loads on the joint surface acting simultaneously with the shear, Sec. 11.8.7.
P_x	The total unfactored vertical design load at and above level x.
Q_D	The effect of dead load.
Q_E	The effect of seismic (earthquake-induced) forces.
Q_L	The effect of live load, reduced as permitted in Sec. 2.1.
Q_S	The effect of snow load, reduced as permitted in Sec. 2.1.
R	The seismic response modification coefficient as given in Table 3-B.
r	A characteristic length of the foundation as defined in Sec. 6.2.1.
r_a	The characteristic foundation length defined by Eq. 6-7.
r_m	The characteristic foundation length as defined by Eq. 6-8.
S	The seismic coefficient for the soil profile characteristics of the site as given in Table 3-A.

S_1, S_2, S_3	The Soil Profile Types as defined in Sec. 3.2.
s_h	Spacing of Special Lateral Reinforcement.
T	The fundamental period of the building as determined in Sec. 4.2.2.
$\tilde{T}$, $\tilde{T}_1$	The effective fundamental period of the building as determined in Sec. 6.2.1 and 6.3.1.
T_a	The approximate fundamental period of the building as determined in Sec. 4.2.2.
T_c	The fundamental period of the component and its attachment.
T_m	The modal period of vibration of the m^{th} mode of the building as determined in Chapter 5.
V_t	The design value of the seismic base shear as determined in Sec. 5.8.
V_x	The seismic shear force at any level as determined in Sec. 4.4 or Sec. 5.8.
$\tilde{V}_1$	The portion of the seismic base shear, $\tilde{V}$, contributed by the fundamental mode, Sec. 6.3.
ΔV	The reduction in V as determined in Sec. 6.2.
ΔV_1	The reduction in V_1 as determined in Sec. 6.3.
v_s	The average shear wave velocity for the soils beneath the foundation at large strain levels, Sec. 6.2.
v_{so}	The average shear wave velocity for the soils beneath the foundation at small strain levels, Sec. 6.2.
W	The total gravity load of the building as defined in Sec. 4.2.
$\bar{W}$	The effective gravity load of the building as defined in Sec. 6.2 and 6.3.
$\bar{W}_m$	The effective modal gravity load determined in accordance with Eq. 5-2.
W_c	The gravity load of a component of the building.
w_i, w_n, w_x	The portion of W that is located at or assigned to level i, n, or x, respectively.
x	The level under consideration; $x = 1$ designates the first level above the base.

α	The relative weight density of the structure and the soil as determined in Sec. 6.2.1.
$\tilde{\beta}$	The fraction of critical damping for the coupled structure-foundation system, determined in Sec. 6.2.1.
β_o	The foundation damping factor as specified in Sec. 6.2.1.
γ	The average unit weight of soil.
Δ	The design story drift as determined in Sec. 4.6.1 or 4.6.2.
Δ_a	The allowable story drift as specified in Sec. 3.8.
Δ_m	The design modal story drift determined in Sec. 5.6.
δ_x	The deflection of level x at the center of the mass at and above level x, Eq. 4-9.
δ_{xe}	The deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Sec. 4.6.1.
δ_{xem}	The modal deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Sec. 5.6.
δ_{xm}, $\tilde{\delta}_{xm}$	The modal deflection of level x at the center of the mass at and above level x as determined by Eq. 5-5 and 6-15.
$\tilde{\delta}_x$, $\tilde{\delta}_{x1}$	The deflection at the center of the mass at and above level x, Eq. 6-11 and 6-14.
θ	The stability coefficient for P-delta effects as determined in Sec. 4.6.2.
κ	The overturning moment reduction factor, Eq. 4-6.
ϕ	The capacity reduction factor.
ϕ_{im}	The displacement amplitude at the i^{th} level of the building for the fixed base condition when vibrating in its m^{th} mode, Sec. 5.5.

Chapter 3

STRUCTURAL DESIGN REQUIREMENTS

3.1 DESIGN BASIS

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

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

3.2 SITE EFFECTS

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

3.2.1 Soil Profile Types

The effects of site conditions on building response shall be established based on soil profile types defined below.

SOIL PROFILE TYPE S_1 is a profile with:

1. Rock of any characteristic, either shale-like or crystalline in nature. Such material may be characterized by a shear wave velocity

greater than 2,500 feet per second or by other appropriate means of classification, or

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

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

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

In locations where the soil properties are not known in sufficient detail to determine the soil profile type or where the profile does not fit any of the three types, Soil Profile S_2 or Soil Profile S_3 shall be used depending on whichever soil profile type results in the higher value of seismic coefficient, C_s, as determined in Sec. 4.2.1.

3.2.2 Site Coefficient

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

3.2.3 Soil-Structure Interaction

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

3.3 FRAMING SYSTEMS

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

3.3.1 Classification of Framing Systems

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

3.3.2 Combinations of Framing Systems

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

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

EXCEPTION:

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

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

3.3.3 Seismic Performance Categories A and B

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

3.3.4 Seismic Performance Category C

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

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

1. Moment resisting frame system with Special Moment Frames.

2. A Dual System utilizing Special Moment Frames.

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

a. 60 percent when the braced frame or shear walls are arranged only on the perimeter,

b. 40 percent when some of the braced frames or shear walls are arranged on the perimeter,

c. 30 percent for other arrangements.

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

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

C. Deformational Compatability. Every structural component not included in the seismic force resisting system in the direction under consideration shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the design story drift Δ as determined in accordance with Sec. 4.6.

D. Special Moment Frames. A Special Moment Frame that is used but not required by these provisions may be discontinued and supported by a more rigid system with a lower R value subject to the requirements in Sec. 3.7.3.

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

3.3.5 Seismic Performance Category D

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

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

3.4 BUILDING CONFIGURATION

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

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

3.4.1 Plan Configuration

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

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

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

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

3.4.2 Vertical Configuration

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

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

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

3.5 ANALYSIS PROCEDURES

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

3.5.1 Seismic Performance Category A

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

3.5.2 Seismic Performance Category B

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

3.5.3 Seismic Performance Categories C and D

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

3.6 DESIGN AND DETAILING REQUIREMENTS

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

3.6.1 Seismic Performance Category A

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

3.6.2 Seismic Performance Category B

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

A. Components. Components of the seismic resisting system and other structural components shall conform to the requirements of Sec. 3.7 (except Sec. 3.7.3 and 3.7.12) and to Sec. 7.4.

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

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

3.6.3 Seismic Performance Category C

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

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

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

3.6.4 Seismic Performance Category D

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

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

3.7 STRUCTURAL COMPONENT LOAD EFFECTS

In addition to the evaluation required by the governing Building Code for other load combinations, all building components shall be provided with strengths sufficient to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, live, and snow loads. The direction of application of seismic forces used in design shall be that which will produce the most critical load effect in each component. The second-order effects shall be included where applicable.

3.7.1 Combination of Load Effects

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

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

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

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

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

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

3.7.2 Orthogonal Effects

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

EXCEPTION:

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

3.7.3 Discontinuities in Strength of Vertical Resisting System

The design of buildings assigned to Seismic Performance Category C or D shall consider the potential for adverse effects when the ratio of the strength provided in any story to the strength required is significantly less than that ratio for the story immediately above and the strengths shall be adjusted to compensate for this effect.

3.7.4 Nonredundant Systems

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

3.7.5 Ties and Continuity

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

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

3.7.6 Concrete or Masonry Wall Anchorage

Concrete and masonry walls shall be anchored to the roof and all floors that provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting a seismic lateral force, F_p, induced by the wall but not less than a force of 1,000 A_v (lb) per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

3.7.7 Anchorage of Nonstructural Systems

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

3.7.8 Collector Elements

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

3.7.9 Diaphragms

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

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

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

3.7.10 Bearing Walls

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

3.7.11 Inverted Pendulum-Type Structures

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

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

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

3.8 DEFLECTION AND DRIFT LIMITS

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

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The design story drift, Δ, as determined in Sec. 4.6 or 5.8, shall not exceed the allowable story drift Δ_a as obtained from Table 3-C for any story. Single-story buildings in Seismic Hazard Exposure Group I that are constructed with nonbrittle finishes and whose seismic resisting system is not attached to equipment or processes need not meet the drift requirement in Table 3-C. For structures with significant torsional deflections, the effect of maximum drift, including torsional effects, shall be considered for stability and damage control.

TABLE 3-A Soil Profile Coefficient

	Soil Profile Type			
	S_1	S_2	S_3	S_4
S	1.0	1.2	1.5	2.0

Insert Table from TC 3
(with text and numbers)

TABLE 3-B Response Modification Coefficients[a]

Type of Structural System	Vertical Seismic Resisting System	Coefficients R^h	$C_d{}^i$
BEARING WALL SYSTEM: A structural system with bearing walls providing support for all or major portions of the vertical loads.	Light framed walls with shear panels	6-1/2	4
Seismic force resistance is provided by shear walls or braced frames.	Reinforced concrete shear walls	4-1/2	4
	Reinforced masonry shear walls[g]	3-1/2	3
	Braced frames	4	3-1/2
	Unreinforced masonry shear walls[g]	1-1/4	1-1/4
BUILDING FRAME SYSTEM: A structural system with essentially complete Space Frame providing support for vertical loads.	Light framed walls with shear panels	7	4-1/2
Seismic force resistance is provided by shear walls or braced frames.	Reinforced concrete shear walls	5-1/2	5
	Reinforced masonry shear walls[g]	4-1/2	4
	Braced frames	5	4-1/2
	Eccentrically Braced Frame	8	5-1/2
	Unreinforced masonry shear walls[g]	1-1/2	1-1/2

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TABLE 3-8 continued

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[h]	C_d[i]
MOMENT RESISTING FRAME SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. Seismic force resistance is provided by Ordinary or Special Moment Frames capable of resisting the total prescribed forces.	Special Moment Frames of steel[c]	8	5-1/2
	Special Moment Frames of reinforced concrete[d]	8	5-1/2
	Ordinary Moment Frames of steel[b]	4-1/2	4
	Ordinary Moment Frames of reinforced concrete[e]	2 4	2
	Intermediate Moment Frames of reinforced concrete[f]	4	3-1/2
DUAL SYSTEM: A structural system with an essentially complete Space Frame providing support for vertical loads. A Special Moment Frame shall be provided that shall be capable of resisting at least 25 percent of the prescribed seismic forces. The total seismic force resistance is provided by the combination of the Special Moment Frame and shear walls or braced frames in proportion to their rigidity.	Reinforced concrete shear walls	8	6-1/2
	Reinforced masonry shear walls[g]	6-1/2	5-1/2
	Wood sheathed shear panels	8	5
	Braced frames	6	5
A structural system with an intermediate moment frame of reinforced concrete <u>or</u> an ordinary moment frame of steel in combination with the systems shown.	Reinforced concrete shear walls	6	5
	Reinforced masonry shear walls[g]		
	Wood shear panels	7	4-1/2
	Braced frames	5	4-1/2

TABLE 3-B continued

Type of Structural System	Vertical Seismic Resisting System	Coefficients R[h]	C_d[i]
INVERTED PENDULUM STRUCTURES: Structures where the framing resisting the total prescribed seismic forces acts essentially as isolated cantilevers and provides support for vertical load.	Special Moment Frames of structural steel[c]	2-1/2	2-1/2
	Special Moment Frames of reinforced concrete[d]	2-1/2	2-1/2
	Ordinary Moment Frames of structural steel[b]	1-1/4	1-1/4

N/A = Not applicable.

[a]These values are based on best judgment and data available at time of writing and need to be reviewed periodically.
[b]As defined in Sec. 10.4.1.
[c]As defined in Sec. 10.6.
[d]As defined in Sec. 11.5.
[e]As defined in Sec. 11.3.
[f]As defined in Sec. 11.4.
[g]As defined in Sec. 12.3.
[h]Coefficient for use in Eq. 4-2, 4-3, and 5-3.
[i]Coefficient for use in Eq. 4-9.

TABLE 3-C Allowable Story Drift Δ_a

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

[a]For buildings three stories or less in height and where there are no brittle finishes, this limit may be increased by one-third. For one-story buildings where there are no brittle finishes and where the seismic resisting system is not attached to equipment or processes, this limit may be exceeded.

[b]For one-story steel buildings where there are no brittle finishes and where it can be demonstrated that greater drifts can be tolerated by all components, including nonstructural ones, this limit may be exceeded.

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Chapter 4

EQUIVALENT LATERAL FORCE PROCEDURE

4.1 GENERAL

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

4.2 SEISMIC BASE SHEAR

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

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

where

C_s = the seismic design coefficient, and

W = the total gravity load of the building.

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

1. Partitions and permanent equipment including operating contents.

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

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

The value of C_s may be determined in accordance with Eq. 4-2, 4-3, or 4-3a, as appropriate. Eq. 4-2 requires calculation of the fundamental period of the building as specified in Sec. 4.2.2. For low buildings or in other instances when it is not desired to calculate the period of the building, C_s shall be determined using Eq. 4-3 or 4-3a, as appropriate.

4.2.1 Calculation of Seismic Coefficient

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

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

where

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

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

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

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

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

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

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

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

where

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

EXCEPTION:

For Soil Profile Type S_3 and S_4 in areas where $A_a \geqslant 0.30$, C_s shall be determined in accordance with the following:

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

4.2.2 Period Determination

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

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

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

where

C_T = 0.035 for steel frames,

C_T = 0.030 for concrete frames, and

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

2. For all other buildings:

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

where

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

4.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

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

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

where

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

w_i and w_x are the portion of W located at or assigned to level i or x; h_i and h_x are the height above the base to level i or x; and k is an exponent related to the building period as follows:

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

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

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

4.4 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

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

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

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

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

addition
See TC2-14

4.5 OVERTURNING

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

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

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

where

κ = 1.0 for the top 10 stories,

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

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

The foundations of buildings, except inverted pendulum structures, may be designed for the foundation overturning design moment, M_f, at the foundation-soil interface determined using Eq. 4-8 with κ = 0.75 for all building heights.

4.6 DRIFT DETERMINATION AND P-DELTA EFFECTS

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

4.6.1 Story Drift Determination

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

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

where

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

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

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

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

4.6.2 P-Delta Effects

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

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

where

Δ = the design story drift,

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

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

P_x = the total unfactored vertical design load at and above level x.

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

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

TABLE 4-A Coefficient for Upper Limit on Calculated Period

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

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

Chapter 5

MODAL ANALYSIS PROCEDURE

5.1 GENERAL

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

5.2 MODELING

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

5.3 MODES

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

5.4 PERIODS

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

5.5 MODAL BASE SHEAR

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

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

where

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

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

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

where

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

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

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

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

EXCEPTIONS:

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

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

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

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

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

5.6 MODAL FORCES, DEFLECTIONS, AND DRIFTS

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

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

and

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

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

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

where

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

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

5.7 MODAL STORY SHEARS AND MOMENTS

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

5.8 DESIGN VALUES

The design value for base shear, each of the story shear, moment and drift quantities, and the deflection at each level shall be determined by combining their modal values, obtained from Sec. 5.6 and 5.7. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values. The design base shear, V_t, shall be compared with a base shear, $\bar{V}$, calculated using a period $T = 1.2\ C_a T_a$ where C_a is given in Table 4-A. Where V_t is less than $\bar{V}$, the design story shears, moments and drifts, and floor deflections shall be multiplied by $\bar{V}/V_t$. The base shear need not exceed the values determined in accordance with Sec. 4.2.

5.9 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

The provisions of Sec. 4.4 apply.

5.10 FOUNDATION OVERTURNING

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

5.11 P-DELTA EFFECTS

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

Chapter 6

SOIL-STRUCTURE INTERACTION

6.1 GENERAL

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

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

6.2 EQUIVALENT LATERAL FORCE PROCEDURE

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

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6.2.1 Base Shear

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

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

The reduction, ΔV, shall be computed in accordance with the following:

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

where

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

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

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

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

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

A. Effective Building Period. The effective period, $\tilde{T}$, shall be determined in accordance with the following:

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

where

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

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

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

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

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

g = the acceleration of gravity.

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

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

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

γ = the average unit weight of the soils.

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

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

where

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

$$\alpha = \bar{W}/\gamma A_o\bar{h}. \tag{6-6}$$

r_a and r_m = characteristic foundation lengths, defined by:

$$r_a = \sqrt{A_o/\pi}, \tag{6-7}$$

$$r_m = \sqrt[4]{4I_o/\pi}. \tag{6-8}$$

A_o = the area of the foundation.

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

B. Effective Damping. The effective damping factor for the structure-foundation system, $\tilde{\beta}$, shall be computed from the following:

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

where

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

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

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

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

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

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

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

where

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

A_o = the area of the load-carrying foundation, and

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

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

EXCEPTION:

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

$$\beta_o' = (4D_s/V_s\tilde{T})^2\,\beta_o, \tag{6-10}$$

if $4D_s/v_s\tilde{T} < 1$ where D_s is the total depth of the stratum.

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

6.2.2 Vertical Distribution of Seismic Forces

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

6.2.3 Other Effects

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

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

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

where

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

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

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

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

6.3 MODAL ANALYSIS PROCEDURE

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

6.3.1 Modal Base Shears

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

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

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

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

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

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

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

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

6.3.2 Other Modal Effects

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

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

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

and

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

where

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

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

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

6.3.3 Design Values

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

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

TABLE 6-A Values of G/G_o and v_s/v_{so}

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

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

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

γ = the average unit weight of the soils.

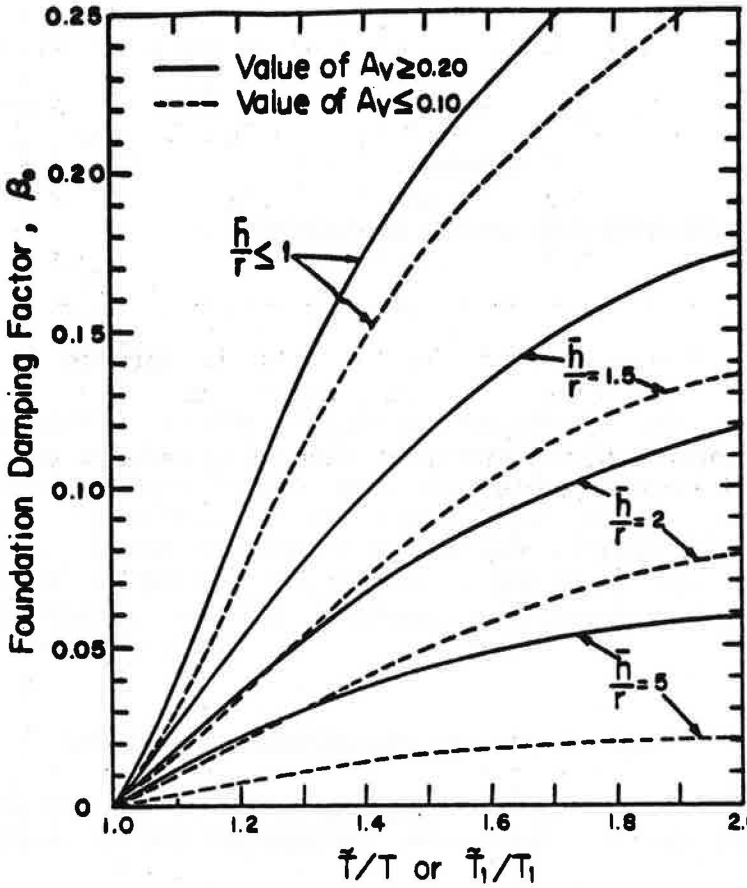

FIGURE 6-1 Foundation damping factor

Chapter 7

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL

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

7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS

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

7.2.1 Structural Materials

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

7.2.2 Soil Capacities

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

7.3 SEISMIC PERFORMANCE CATEGORY A

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

7.4 SEISMIC PERFORMANCE CATEGORY B

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

7.4.1 Investigation

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

7.4.2 Pole-Type Structures

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

7.4.3 Foundation Ties

never map values A & V not A_a & A_v !

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

very dense granular soils or hard cohesive soils,

7.4.4 Special Pile Requirements

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

The piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap for a distance equal to the development length as specified in Chapter 11. 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 for compression without reduction in length for excess area. Where special reinforcement at the top of the pile is required, alternative measures for containing concrete and maintaining ductility at the top of the pile will be permitted provided due consideration is given to forcing the hinge to occur in the contained section.

A. Uncased Concrete Piles. Reinforcing steel shall be provided in the top portion of uncased cast-in-place concrete piles or caissons for a distance of 10 pile diameters with a minimum steel ratio of 0.0025 with a minimum of four No. 5 bars. Ties (or equivalent spirals) shall be provided at 16-bar-diameter spacing (maximum spacing) with a maximum spacing of 4 inches in the top 2 feet.

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

C. Steel Pipe Piles. Reinforcement equal to 1 percent of the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment.

D. Precast Concrete Piles. Longitudinal reinforcing steel shall be provided for precast concrete piles with a minimum steel ratio of 0.01.

E. Precast-Prestressed Piles. The upper 2 feet of the pile shall have No. 3 ties minimum at not over 4-inch spacing or equivalent spirals. The pile cap connection may be by means of dowels as required in Sec. 7.4.4.C. Pile cap connection may be by means of developing pile reinforcing strand if a ductile connection is provided.

7.5 SEISMIC PERFORMANCE CATEGORY C

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

7.5.1 Investigation

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

7.5.2 Foundation Ties

sec end

Individual spread footings, unless founded directly on rock, as defined in Sec. 3.2.1.1, shall be interconnected by ties. All ties shall be capable of carrying, in tension or compression, a force equal to $A_v/4$ of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by reinforced beams within slabs on grade or confinement by competent rock, or other approved means.

to very dense granular soils or hard cohesive soils.

7.5.3 Special Pile Requirements

The following special requirements shall apply:

A. Uncased Concrete Piles. Reinforcing steel shall be provided for uncased cast-in-place concrete piles, drilled piers, or caissons with a minimum steel ratio of 0.005 with a minimum of four No. 6 bars. Ties shall be provided at 8-bar-diameter spacing with a maximum spacing of 3 inches in the top 4 feet. Ties shall be a minimum of No. 3 bars for up to 20-inch diameter piles and No. 4 bars for piles of larger diameter.

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

C. Precast Concrete Piles. Ties in precast concrete piles shall conform to the requirements of Chapter 11 for the top half of the pile. Precast concrete and prestressed concrete piling shall be designed to withstand maximum imposed curvatures resulting from soil deformation. Pile cap connection shall not be made by developing exposed strand.

D. Steel Piles. The connection between the pile cap and steel piles or unfilled steel pipe piles shall be designed for a tensile force equal to 10 percent of the pile compression capacity.

E. Precast-Prestressed Piles.

1. For the body of fully embedded foundation piling subjected to vertical loads only, or where the design bending moment does not exceed 0.20 M_{nb} (where M_{nb} is the unfactored ultimate moment capacity at balanced strain conditions as defined in Reference 11.1, Sec. 10.3.2), spiral reinforcing shall be provided such that $\rho_s \geqslant 0.006$ (0.2 percent).

2. For free-standing piling and hollow-core or marine piling subject to severe installation and operational forces, spiral reinforcing shall be provided such that $\rho_s \geqslant 0.022$ (0.7 percent) or a spacing satisfying the following relationship if it results in a percentage of spiral greater than that given above:

$$S_{sp} = f_y A_{sp}/(C + 7\ d_b) f_r,$$

where

S_{sp} = spacing of spiral reinforcing,

f_y = yield strength of spiral reinforcing,

A_{sp} = area of spiral reinforcing,

C = cover over the spiral reinforcing,

d_b = diameter of spiral reinforcing,

f_r = modulus of rupture of concrete, and

ρ_s = ratio of volume of spiral reinforcing to total volume of core (out-to-out of spirals) and not less than that given in Chapter 11.

3. Any piling installed in layered soils imposing severe curvatures during earthquake shall have the same amount of spiral reinforcing indicated in Item 2 above, accompanied by additional amounts of flexural reinforcing indicated by moment-curvature relationships developed for the pile and soil profile present.

4. The top and bottom portion of hollow core piling and rigid frame piling where high values of shear and moment occur simultaneously should contain spiral reinforcing with $\rho_s \geqslant 0.031$ (1.0 percent) for a distance of two pile diameters or two times the width of the pile.

7.6 SEISMIC PERFORMANCE CATEGORY D

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

7.6.1 Special Pile Limitations

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

deletion recommended by Soils engr. from Washington

Chapter 8

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS AND SYSTEMS

8.1 GENERAL REQUIREMENTS

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

EXCEPTIONS:

1. Those systems or components designated in Table 8-B or 8-C for L peformance level that are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with a Seismicity Index of 1 or 2 or that are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with a Seismicity Index of 1 are not subject to the provisions of this chapter.

2. Where alterations or repairs are made, the forces on systems or components in existing buildings may be modified in accordance with Part 3 of these provisions.

3. Elevator systems that are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with a Seismicity Index of 1 or 2 or that are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with a Seismicity Index of 1 are not subject to the provisions of this chapter.

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

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

8.1.1 Interrelationship of Components

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

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

TC8-1

8.1.2 Connections and Attachments

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

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

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

8.1.3 Performance Criteria

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

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

8.2 ARCHITECTURAL DESIGN REQUIREMENTS

8.2.1 General

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

8.2.2 Forces

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

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

where

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

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

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

A_v = The seismic coefficient representing the greater of the Effective Peak Velocity-Related Acceleration or the Effective Peak Acceleration A_a as determined in Sec. 1.4, and

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

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

EXCEPTIONS:

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

8.2.3 Exterior Wall Panel Attachment

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

8.2.4 Component Deformation

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

EXCEPTION:

Components assigned an L performance factor in Table 8-B may provide for a design story drift of Δ/2.

8.2.5 Out-of-Plane Bending

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

8.3 MECHANICAL AND ELECTRICAL DESIGN REQUIREMENTS

8.3.1 General

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

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

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

8.3.2 Forces

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

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

where

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

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

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

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

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

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

where

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

h_n = the height above the base to level n.

The attachment amplification factor, a_c, shall be determined as follows:

For fixed or direct attachment to buildings	$a_c = 1$
For resilient mounting system With seismic activated restraining device	$a_c = 1$
With elastic restraining device	
If $T_c/T < 0.6$ or $T_c/T > 1.4$	$a_c = 1$
If $T_c/T \geqslant 0.6$ or $\leqslant 1.4$	$a_c = 2$ minimum[4]
If mounted on the ground or on a slab in direct contact with the ground	$a_c = 2$

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

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

$$T_c = 0.32 \sqrt{W_c/K}, \qquad (8\text{-}4)$$

where K is the stiffness of the equipment support attachment determined in terms of load per unit deflection of the center of gravity (lb/in.) as follows:

For stable resilient attachments, K = spring constant.

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

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

8.3.3 Attachment Design

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

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[4]See Chapter 8 Commentary.

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

8.3.4 Component Design

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

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

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

8.3.5 Utility and Service Interfaces

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

8.4 ELEVATOR DESIGN REQUIREMENTS

8.4.1 Reference Document

The design and construction of elevators and components shall conform to the requirements of ANSI/ASME A17.1-1984, American National Standard Safety Code for Elevators and Escalators, including Appendix F, Recommended Elevator Safety Requirements for Seismic Risk Zone 3 or Greater, except as modified by provisions of this chapter.

8.4.2 Elevators and Hoistway Structural System

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

W_c is defined as follows:

Element = W_c,

Traction Car = C + 0.4 L,

Counterweight = W, and

Hydraulic = C + 0.4 L + 0.25 P,

where

C = the weight of the car,

L = rated capacity,

W = the weight of counterweight, and

P = the weight of plunger.

8.4.3 Elevator Machinery and Controller Anchorage(s)

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

8.4.4 Seismic Controls

All elevators with a speed of 150 fpm or greater shall be furnished with the following signaling devices: (a) a seismic switch device to provide an electrical alert or command for the safe automatic emergency operation of the elevator system, and (b) a counterweight displacement or derailment device to detect lateral motion of the counterweight.

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

8.4.5 Retainer Plates

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

8.4.6 Deflection Criteria

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

TC8-3 8.5 Base Isolation

8.5.1 Displacement

The anticipated maximum - - - etc.

8.6 Raised Access Floors

TC8-4

8.6.1 Design Loads

Allowance shall be included for the weight of computer and other equipment mounted to the floor.

TABLE 8-A Performance Criteria

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

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

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

Architectural Components	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels III	II	I
Appendages				
Exterior nonbearing walls	0.9	S	S G[b]	S L[d]
Wall attachments	3.0	S	~~G[b]~~	~~L[d]~~
Connector fasteners	6.0		S	S
Veneer attachments	3.0	G	G[a]	L
Roofing units	0.6	G	G[b]	NR
Containers and miscellaneous components (free standing)	1.5	G	G	NR
Raised Access Floors	2.0	S	S	S
Partitions				
Stairs and shafts	1.5	S	G[c]	G
Elevator shafts	1.5	S	L[c]	L[e]
Vertical shafts	0.9	S	L[c]	L[f]
Horizontal exits including ceilings	0.9	S	S	G
Public corridors	0.9	S	G	L
Private corridors	0.6	S	L	NR
Full-height area separation partitions	0.9	S	G	G
Full-height other partitions	0.6	S	L	L
Partial-height partitions	0.6	G	L	NR
Structural fireproofing	0.9	S	G[c]	L[f]
Ceilings				
Fire-rated membrane	0.9	S	G[c]	G
Nonfire-rated membrane	0.6	G	G	L
Architectural equipment--ceiling, wall, or floor mounted	0.9	S	G	L
Architectural components--elevator and hoistway structural systems				
Structural frame providing supports for guiderail brackets	1.25	S	G	G
Guiderails and brackets	1.25	S	G	G
Car and counterweight guiding members	1.25	S	G	G

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TABLE 8-B Continued

NR = not required.

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

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

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

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

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

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

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

Mechanical/Electrical Components[a]	C_c Factor	Seismic Hazard Exposure Group Required Performance Characteristic Levels III	II	I
Emergency electrical systems (code required)				
Fire and smoke detection system (code required)	2.00	S	S	S
Fire suppression systems (code required)				
Life safety system components				
Elevator machinery and controller anchorage	1.25	S	G	G
Boilers, furnaces, incinerators, water heaters, and other equipment using combustible energy sources or high-temperature energy sources, chimneys, flues, smokestacks and vents				
Communication systems				
Electrical bus ducts and primary cable systems	2.00	S	G	L
Electrical motor control centers, motor control devices, switchgear, transformers, and unit substations				
Reciprocating or rotating equipment				
Tanks, heat exchangers, and pressure vessels				
Utility and service interfaces				
Machinery (manufacturing and process)	0.67	S	G	L
Lighting fixtures	0.67[c]	S	G	L
Ducts and piping distribution systems				
Resiliently supported	2.00	S	G	NR
Rigidly supported	0.67[d]	S	G	NR

Building-operational computer equipment

TCB-6

TABLE 8-C Continued

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

NR = not required.

[a]Where mechanical or electrical components are not specifically listed in Table 8-C, the designer shall select a similarly listed component, subject to the approval of the authority having jurisdiction, and shall base the design on the performance and C_c values for the similary component.

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

[c]Hanging- or swinging-type fixtures shall use a C_c value of 1.5 and shall have a safety cable attached to the structure and the fixture at each support point capable of supporting four times the vertical load.

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

(1) gas piping less than 1 inch inside diameter,
(2) piping in boiler and mechanical rooms less than 1-1/4 inches inside diameter,
(3) all other piping less than 2-1/2 inches inside diameter,
(4) all electrical conduit less than 2-1/2 inches inside diameter,
(5) all rectangular air-handling ducts less than 6 square feet in cross-sectional area,
(6) all round air-handling ducts less than 28 inches in diameter,
(7) all piping suspended by individual hangers 12 inches or less in length from the top of the pipe to the bottom of the support for the hanger, and
(8) all ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

Chapter 9

WOOD

9.1 REFERENCE DOCUMENTS

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

TC7-9-1

Ref. 9.1	National Design Specification for Stress Grade Lumber and Its Fastenings	NDS (1982)
Ref. 9.2	American Softwood Lumber Standard	PS 20 (1970)
Ref. 9.3	Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber	ASTM D245 (1981) ~~(1981)~~ 1986
Ref. 9.4	Methods for Establishing Clear Wood Strength Values	ASTM D2555 (1981)
Ref. 9.5	Softwood Plywood - Construction and Industrial	PS 1 (1983)
Ref. 9.6	Mat Formed Wood Particle Board	ANSI A208.1 (1979)
Ref. 9.7	Preservative Treatment by Pressure Process Pressure Treatment of Timber Products	AWPA C1 (1982), 1985 (1988) C2 1983), C3 (1981), (1986) C9 (1979), and C29 (1982) ASTM D1760-83
Ref. 9.8	American National Standard for Wood Products--Structural Glued Laminated Timber	ANSI/AITC A190.1 (1983)
Ref. 9.9	Design and Manufacturing Standard Specification for Structural Glued Laminated Timber of Softwood Species	AITC 117 (1984) (1985)
Ref. 9.10	Wood Poles	ANSI 05.1 (1979) (1987)

Ref. 9.11	Round Timber Piles	ASTM D25 (1979) (1986)
Ref. 9.12	One- and Two-family Dwelling Code International Conference of Building Officials Building Officials and Code Administrators Southern Building Code Congress	 1983 1983 1983
Ref. 9.13	Gypsum Wallboard	ASTM C36-64 (1980)
Ref. 9.14	~~Fiberboard, Insulating~~ Cellulosic Fiberboard	~~ASTM D2277-64T (1980)~~ ANSI/AHA A 194.1 (1985)
Ref. 9.15	Plywood Design Specifications	APA (1983)
Ref. 9.16	Plywood Diaphragm Construction	APA (1978)

9.2 STRENGTH OF MEMBERS AND CONNECTIONS

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

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

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

9.3 SEISMIC PERFORMANCE CATEGORY A

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

9.3.1 Bracing Requirements

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

9.4 SEISMIC PERFORMANCE CATEGORY B

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

9.4.1 Detailing Requirements

The construction shall comply with the requirements given below.

A. Anchorage of Concrete or Masonry Walls. The diaphragm sheathing shall not be used for providing ties and splices required in Sec. 3.7.5 and 3.7.6.

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

9.5 SEISMIC PERFORMANCE CATEGORY C

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

9.5.1 Material Limitations

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

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

9.5.2 Framing Systems

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

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

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

C. Conventional Light Frame Construction. Buildings over one story in height of conventional light frame construction shall have solid sheathing of one of the materials specified in Sec. 9.7.3.A or

9.7.3.B applied for the full height over at least 40 percent of the length of the building at each exterior wall of the stories below the top story.

9.5.3 Detailing Requirements

Special details for Category C construction are given below.

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

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

9.6 SEISMIC PERFORMANCE CATEGORY D

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

9.6.1 Material Limitations

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

9.6.2 Framing Systems

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

9.6.3 Diaphragm Limitations

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

9.7 CONVENTIONAL LIGHT TIMBER CONSTRUCTION

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

9.7.1 Wall Framing and Connections

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

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

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

C. Bottom Plates. Studs shall have full bearing on a plate or sill of not less than 2 inch nominal thickness and having a width at least equal to the width of the studs unless specifically excepted in Sec. 9.7.3.

9.7.2 Wall Sheathing Requirements

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

9.7.3 Acceptable Types of Wall Sheathing

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

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

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

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

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

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

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

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

9.8 ENGINEERED TIMBER CONSTRUCTION

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

9.8.1 Framing Requirements

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

9.8.2 Requirements for All Shear Panels

Horizontal and vertical shear panels shall conform to the requirements in this section and to the requirements in the following section pertaining to the particular type of panel. The shear values may be doubled when identical materials are applied to both sides of the wall.

A. Framing. All framing members used in shear panel construction shall be at least 2 inches in nominal thickness. All boundary members, chords, and collector members of shear walls and diaphragms shall be designed and detailed to transmit the induced axial forces. The boundary members shall be tied together at all corners.

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

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

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

9.8.3 Diagonally Sheathed Shear Panels

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

A. Conventional Construction. Such lumber shear panels shall be made up of 1 inch nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards shall be nailed to each intermediate bearing member with not less than two 8d nails for 1 by 6 nominal boards and three 8d nails for 1 by 8 or wider boards. One additional nail shall be provided in each board at shear panel boundaries. Where box nails are used, one additional nail shall be used at each bearing and two additional nails shall be used at shear panel boundaries. End joints in adjacent boards shall be separated by at least one stud or joist space and there shall be at least two boards between joints on the same support. Wood shear panels made up of 2-inch-thick diagonal sheathing using 16d nails may be used at the same shear values and in the same locations as for 1-inch boards provided there are no splices in adjacent boards on the same support and the supports are not less than 4 inch nominal depth or 3 inch nominal thickness.

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

B. Special Construction. Special diagonally sheathed shear panels shall conform to conventional diagonally sheathed shear panel construction and the requirements below.

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

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

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

9.8.4 Plywood Shear Panels

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

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

Plywood panels less than 12 inches wide shall be blocked.

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

9.8.5 Shear Panels Sheathed with Other Materials

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

The shear values tabulated shall not be cumulative with the shear value of other materials applied to the same wall.

9.8.6 Detailing Requirements

Concrete or masonry walls shall be anchored to all floors and roofs for the forces prescribed in Sec. 3.7.6. Such anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal and wood ledgers shall not be used in cross-grain bending or tension.

Replace with new table

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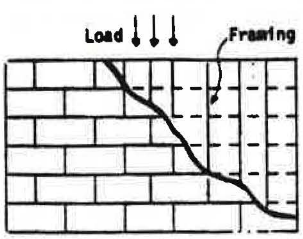
TABLE 9-A Allowable Shear for Seismic Forces in Pounds per Foot for Horizontal Plywood Diaphragms with Framing of Douglas Fir-Larch or Southern Pine[a]

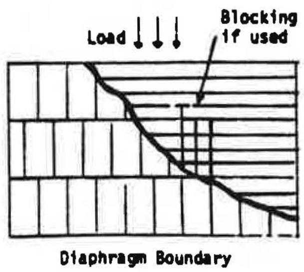
Plywood Grade	Comm. Nail Size	Nominal Penetration in Framing (Inches)	Minimum Nominal Plywood Thickness (Inches)	Minimum Nominal Width of Framing Members (Inches)	Blocked Diaphragms: Nail Spacing at Diaphragm Boundaries (All Cases) At Continuous Panel Edges Parallel to Load (Cases 3 & 4) and at All Panel Edges (Cases 5 & 6): 6	4	2-1/2[b]	2[b]	Unblocked Diaphragms: Nails Spaced 6" Maximum at Supported End: Load Perpendicular to Unblocked Edges and Continuous Panel Joints (Case 1)	All Other Configurations (Cases 2, 3, and 4)
					Nail Spacing at Other Plywood Panel Edges: 6	6	4	3		
Structural I	6d	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	15/32	2	320	425	640[b]	730[b]	285	215
				3	360	480	720	820	320	240
C-D, C-C Structural II and Other Similar Grades	6d	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			15/32	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	15/32	2	290	385	575[b]	655[b]	255	190
				3	325	430	650	735	290	215
			19/32	2	320	425	640[b]	730[b]	285	215
				3	360	480	720	820	320	240

[a]Space nails 10 inches on center for floors and 12 inches on center for roofs along the intermediate framing members. Allowable shear values for nails in framing members or other species set forth in Table 8.1A NDS (Ref. 9.1) shall be calculated for all grades by multiplying the values for nails in Structural I by the following factors: Group III, 0.82, and Group IV, 0.65.

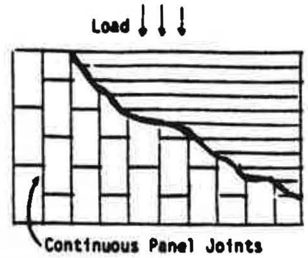
[b]Framing shall be 3-inch nominal or wider and nails shall be staggered where nails are spaced 2 inches or 2-1/2 inches on center and where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches on center.



CASE 1



CASE 2



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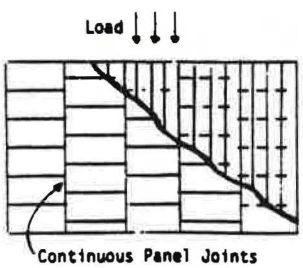

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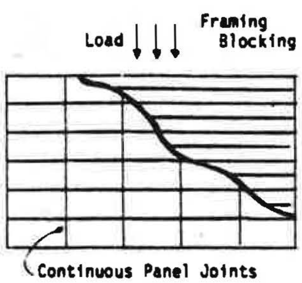

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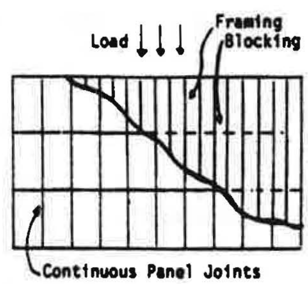

CASE 6

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

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

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

[a]All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch plywood installed with face grain parallel to studes spaced 24 inches on center and 12 inches on center for other conditions and plywood thicknesses. Allowable shear values for nails in framing members of other species set forth in Table 8.1A NDS (Ref. 9.1) shall be calculated for all grades by multiplying the values for common and galvanized nails in Structural I and galvanized casing nails in other grades by the following factors: Group III, 0.82, and Group IV, 0.65.

[b]Nail spacing at plywood panel edges.

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

TABLE 9-8 Continued

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[d]Common or galvanized box nails.

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

[f]Galvanized casing nails.

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

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

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

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

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

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

TABLE 9-D Continued

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

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

Chapter 10

STEEL

10.1 REFERENCE DOCUMENTS

The quality and testing of steel materials and the design and construction of steel components that resist seismic forces shall conform to the requirements of the refereces listed in this section except as modified by provisions of this chapter.

Ref. 10.1 The American Institute of Steel Construction (AISC) Specifications (Parts 1 and 2) for the Design, Fabrication and Erection of Structural Steel for Buildings, November 1, 1978 (TC6-1)
Including Supplement No 1 effective March 11, 1986

Ref. 10.2 The Specification for the Design of Cold-formed Steel Structural Members, American Iron and Steel Institute (AISI), with Addendum No. 1, September 3, 1980 Aug 10, 1986 (TC6-2)

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

Ref. 10.4 The Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders, Steel Joist Institute, 1984 Edition (TC6-3)
1986

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

10.2 STRENGTH OF MEMBERS AND CONNECTIONS

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

The value of ϕ shall be as follows:

Members, connections, and base plates that develop the strength of the members or structural systems	$\phi = 0.90$
Connections that do not develop the strength of the member or structural system, including connection of base plates and anchor bolts, or do not conform to Sec. 10.6.6	$\phi = 0.67$

Metal deck diaphragms $\phi = 0.60$

Partial penetration welds in columns when subjected to tension stresses $\phi = 0.80$

10.2.1 Structural Steel

Reference 10.1 shall be modified as follows:

A. Load Combination. AISC Sec. 1.5.6 shall read as follows:

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

B. Euler Stress. AISC Sec. 1.6.1. The definition of F'_e for the purpose of determining the strength of structural steel members shall read as follows:

$$F'_e = \frac{\pi^2 E}{(Kl_b/r_b)^2}$$

(in the expression for F'_e, l_b is the actual length in the plane of bending and r_b is the corresponding radius of gyration. K is the effective length factor in the plane of bending.)

C. Member Strength. Amend first paragraph of AISC Sec. 2.1 by deleting "or earthquake" and adding the following:

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

(KC-4) D. Shear Strength. In AISC Sec. 2.5, substitute 0.60 (or appropriate value from final draft of AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings) for 0.55 in Formula (2.5-1) and delete the word "factored" from the definition of V_u.

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

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

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

10.2.2 Cold Formed Steel

References 10.2 and 10.3 shall be modified as follows:

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

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B. Effective Width. Modify the third paragraph of AISI Sec. 2.3.1.1 of Ref. 10.2 and add to AISI Sec. 2.3.1.1 of Ref. 10.3:

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

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

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

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

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

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

10.2.3 Steel Cables

Reference 10.5, Sec. 5d, shall be modified by substituting 1.5 T_4 when T_4 is the net tension in cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of AISI Sec. 3.1.2 of Ref. 10.6.

10.3 SEISMIC PERFORMANCE CATEGORY A

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

10.4 SEISMIC PERFORMANCE CATEGORY B

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

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10.4.1 Space Frames

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

10.5 SEISMIC PERFORMANCE CATEGORIES C AND D

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

10.5.1 Special Moment Frames

Where a Moment Resisting Frame System is used as the seismic resisting system, it shall be composed of Special Moment Frames conforming to the requirements of Sec. 10.6.

EXCEPTION:

1. Moment frames in one- and two-story buildings assigned to Seismic Performance Category C may be Ordinary Moment Frames.

2. Moment frames in one-story buildings assigned to Seismic Performance Category D may be Ordinary Moment Frames.

10.5.2 Braced Frames

For seismic resisting systems over two stories in height using braced frames, the members shall have a compressive strength equal to at least 50 percent of the required tensile strength.

10.5.3 Eccentrically Braced Frames

10.7 ~~10.6~~ SPECIAL MOMENT FRAME REQUIREMENTS

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

1. Substitute the following for the last three paragraphs of AISC Sec. 2.1:

"Special Moment Frames shall satisfy the requirements for Type 1 construction in the plane of the frame as provided in Sec. 1.2. Type 2 construction is permitted for members between rigid frames. Connections joining a portion of a structure designed on the basis of this Part with a portion not so designed need be no more rigid than ordinary seat-and-cap angle or standard web connections. Except as modified by these rules, all other pertinent provisions of Part 1 shall govern.

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

2. Substitute the following for AISC Sec. 2.2:

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

renumbered 10.7

EXCEPTION:

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

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

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

4. Add the following to AISC Sec. 2.4:

"Column splices shall not be placed in an area in which a potential plastic hinge would form unless the splice fully develops the column section. Partial penetration welds shall not be used for column splices unless it can be shown that the splice strength is adequate to resist load effects of:

a. The plastic capacity of the joints at the end of the column with the yield strengths of members assumed at 1.25 F_y, and

b. The plastic capacity of the joint at one floor, and one-half the plastic capacity of the joint at the other floor with yield strengths of members assumed at F_y, and

c. The load as specified in *NEHRP Recommended Provisions* Eq.3-2a."

5. Add the following to AISC Sec. 2.5:

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

Beam-column joint panel zone areas shall be designed to resist the shears, f_v, based on the capacity of the members framing into the joint, but need not exceed shears produced by deforming the frame two times that resulting from the prescribed forces."

Renumbered 10-7

Applicable definitions of terms remain unchanged.

6. Add the following AISC Sec. 2.8:

"Beam to column connections shall develop the joint capacity determined by the strength of members framing into the joint, unless it can be shown that adequate rotation can be obtained by deformations of the connection materials. The connection consists of only those elements that connect the members of the joint."

Revised TC6-10/10-85

7. Change the start of the second paragraph in AISC Sec. 2.9 to read as follows:

"The foregoing provisions need not apply to members bending about their weak axis. However, in regions not adjacent to a plastic hinge the maximum distance...."

10.8 Concentric Braced Frame Requirement

TC6-10

10.9 Eccentrically Braced Frames
(Popov write up)

TC6-5

Appendix
Load and Resistance Factor Design ← TC 6-6
As an alternative - - etc
TC6-5

Appendix to Chapter 10

ECCENTRICALLY BRACED FRAMES

These tentative provisions for the design of eccentrically braced frames are included for the purpose of completeness. The system is being used but there are no generally accepted design criteria; hence, these provisions are in an appendix. They are believed to be conservative but should be used carefully and with consideration of subsequent research findings. For use with this document, an R value of 7 and C_D value of 5 may be appropriate.

DEFINITIONS

Eccentrically Braced Frame (EBF) is a braced frame in which at least one end of each diagonal brace frames only into a beam and in such a way that at least one ductile link beam is formed in each beam. The beam length outside the link beam region is termed an ordinary beam.

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

Bracing Members are secondary members designed to prevent lateral or torsional buckling of beams in an EBF.

Link Beam End Web Stiffeners are vertical web stiffeners placed on both sides of the web at the intersection of the toe of the diagonal brace and the beam.

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

Link Beam is the part of the beam in an EBF that is designed to yield in shear and/or bending so that buckling or tension failure of the diagonal brace is prevented. When required, the link beam's length is defined by the distance between the toes of diagonal braces or the distance between the toe of a diagonal brace and the column.

ECCENTRICALLY BRACED FRAME REQUIREMENTS

Eccentrically braced frames shall be designed so that in severe earthquakes energy is dissipated through shear and/or flexural yielding in the link beams whereas the diagonal braces and columns shall remain essentially elastic. EBFs shall be designed in accordance with the following provisions:

1. Link beams shall be Grade A36 steel and shall comply with the requirements of Ref. 10.1, Sec. 1.5.1.4.1, except that the flange width-thickness ratio, $b_f/2\ t_f$, shall not exceed 8.5.

2. An EBF bay, deformed as a rigid-ideally plastic mechanism shall be such that the rotation angle between the ordinary beam and the link

beam shall not exceed the rotations specified below at a total story drift of C_D times the drift determined by an elastic analysis at prescribed design forces:

a. Link beams with length of 1.6 M_p/V_p, or less shall have the rotation angle between the beam and the link no greater than 0.06 rad and shall be provided with intermediate web stiffeners as specified in Item 7 below.

b. Link beams of length greater than 1.6 M_p/V_p and less than 2.6 M_p/V_p shall have a rotation angle between the ordinary beam and the link beam no greater than 0.06 rad at 1.6 M_p/V_p and 0.015 rad at 2.6 M_p/V_p. The allowable rotation angle shall vary on a linear basis between 1.6 M_p/V_p and 2.6 M_p/V_p. The link beams shall be provided with intermediate web stiffeners at 6 inches and 12 inches from both ends of the link. Additional intermediate web stiffeners as specified in Item 7 below shall also be provided.

c. Link beams of clear length greater than 2.6 M_p/V_p shall not be used as part of an EBF lateral resisting system. Frame members of clear length greater than 2.6 M_p/V_p shall meet special moment frame (SMRSF) requirements.

3. The flexural plastic strength, M_p, of a link beam shall be computed as the smaller of F_yZ or 1.18 $(F_y - f_a)Z$, where f_a at prescribed design forces is the axial stress in the link beam.

4. The web of the link beam shall be single thickness without doubler plate reinforcement. The web shear at prescribed forces shall not exceed $V_p = 0.55\ F_y d t_w$.

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

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

7. Intermediate link beam web stiffeners required in Item 2 above shall be spaced at intervals not exceeding $(38\ t_w - d/5)$ for a rotation angle of 0.06 rad or $(56\ t_w - d/5)$ for a rotation angle of 0.03 rad or smaller. Interpolation may be used for rotation angles between 0.03 and 0.06 rad.

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

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

10. The beam flanges of link beams connected to a column flange shall have full penetration butt welds to the column and the web connection shall be welded to develop the full shear yield strength of the beam.

11. Where the link beam is connected to the column web, the beam flanges shall have full penetration welds to the connection plates and the web connection shall be welded to develop the full shear yield strength of the beam web. The slope change between the link beam and column shall not exceed 0.015 rad when subjected to the total story drift of C_D times the drift determined by an elastic analysis at prescribed design forces.

12. The strength of each diagonal brace shall be at least 1.5 times the axial force along the diagonal brace needed to reach the yield strength of the link beam. The yield strength shall be the smaller force as determined from V_p defined in Item 4 above or M_p defined in Item 3 above.

13. Columns shall be designed to remain elastic at the yield capacity of the EBF bay as defined in Item 2 above except that F_y for beams shall be increased by a factor of 1.25.

14. Top and bottom flanges of EBF beams shall be laterally braced at the ends of link beams. Bracing shall have a strength to resist at least 1.5 percent of the beam flange strength computed as $F_y b_f t_f$.

15. Top and bottom flanges of the ordinary beam between the bracing members specified in Item 14 above shall be laterally braced at intervals not exceeding 38 r_y. Bracing members shall have a strength to resist 1.0 percent of the beam flange force at the brace point.

16. Ordinary beam connections to columns may be designed as pinned about the strong axis. The connection shall have strength to resist rotation about the longitudinal axis of the beam based on two equal and opposite rotation forces of at least 0.01 $F_y b_f t_f$ acting laterally on the beam flanges.

INSPECTION

Tension groove welded connections between primary members of the frames that are part of the lateral load resisting system shall be tested by nondestructive methods for compliance with AWS D1.1 and job specifications. A program for this testing shall be established by the engineer.

Chapter 11

REINFORCED CONCRETE

11.1 REFERENCE DOCUMENTS

The quality and testing of concrete and steel materials and the design and construction of reinforced concrete components that resist seismic forces shall conform to the requirements of the reference listed in this section except as modified by the provisions of this chapter.

Ref. 11.1 Building Code Requirements for Reinforced Concrete, American Concrete Institute, ACI 318-83, including Appendix A

Ghosh TMCMtg 8/24-25/87

11.1.1 Modifications to Ref. 11.1

No change A. Replace Sec. 9.2.3 of Ref. 11.1 with Sec. 3.7.1 of this document.

" B. Replace Sec. A.2.1.3 and A.2.1.4 of Ref. 11.1 with the provisions of this chapter.

Appendix A

C. Add the following to the end of Sec. A.2.5.1 of Ref. 11.1:

"When reinforcing steel is to be welded, the steel shall comply with ASTM A706. This requirement may be satisfied by the use of steel complying with ASTM A615 provided that this steel meets the carbon equivalent requirements and chemical limits for ASTM A706 steel. Welding shall comply with AWS D1.4-1979, the Structural Welding Code, Reinforcing Steel, published by the American Welding Society. Welding of Stirrups, ties, inserts, or other similar elements to longitudinal reinforcing shall not be permitted." Retain "already in ACI 318"

D. Change Sec. A.3.3.4 of Ref. 11.1 to read:

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

E. Change Sec. A.4.1 of Ref 11.1 to read:

"The requirements of this section apply to Special Moment Frame members having a factored axial compressive force exceeding $A_g f'_c/10$. Frame members shall satisfy the following conditions:...."

(unnecessary)

F. Add the following new paragraph to Sec. A.4.4 of Ref. 11.1:

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

G. Add the following to the end of Sec. A.4.4.5 of Ref. 11.1:

"The special transverse reinforcement shall be placed above the column for at least the development length of the largest vertical reinforcement in the column in accordance with Sec. A.6.4 and below the discontinuity into the foundation for the same development length unless the column terminates on a footing or mat, in which case the special transverse reinforcement shall be extended the compression development length or the lead length of the standard hook."

replace with Blue Book language

H. Add the following to the end of Sec. A.5.1 of Ref. 11.1:

"A cast-in-place topping on a precast floor system may serve as the diaphragm provided the cast-in-place topping is proportioned and detailed to resist the design shear forces. Where untopped precast elements are used for diaphragms, the ϕ factor for connections between elements shall be 0.5." Tie of cont = 0.7

I. Change the last sentence of Sec. A.5.2.3 of Ref. 11.1 to read:

"Stresses <u>may</u> be calculated...."

J. Add the following to the end of Sec. A.5.3.5 of Ref. 11.1:

add sentence: When the

Blue Book '85

"Reinforcement terminating at the edges of shear walls with a minimum edge reinforcement shall have a standard hook enclosing the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups of the same size and spacing as transverse reinforcement."

K. Change the definition of A_j in Sec. A.6.3.1 (and Sec. A.0) of Ref. 11.1 to read:

"Effective cross-sectional area within a joint in a plane parallel to the axis of the reinforcement generating the shear force, not be greater than the column width times the effective depth to the centroid of the longitudinal column reinforcement. Where a girder frames into a support of larger width, effective column width shall not exceed the width of the girder plus the overall depth of the joint."

Replace with ACI 318 language

L. Change the reference to Sec. 9.2 in Sec. A.7.1.3 of Ref. 11.1 to the load combination specified in Sec. 3.7.1 of this document for earthquake sources.

M. Delete the following from Sec. A.7.2.1 of Ref. 11.1:

"...in which the earthquake induced shear force calculated in accordance with Sec. A.7.1.1 represents one-fourth or more of the total design shear,...."

N. Revise Sec. A.8.1 of Ref. 11.1 to read:

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

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

11.2 STRENGTH OF MEMBERS AND CONNECTIONS

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

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

11.3 ORDINARY MOMENT FRAMES

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

11.4 INTERMEDIATE MOMENT FRAMES

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

Change the title of Sec. A.9 of Ref. 11.1 to read: "Requirements for Intermediate Ductility Frames."

11.5 SPECIAL MOMENT FRAMES

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

11.6 SEISMIC PERFORMANCE CATEGORY A

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

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

11.7 SEISMIC PERFORMANCE CATEGORY B

For reinforced concrete construction, Seismic Performance Category B is divided into two classes, B.1 and B.2, as defined in Table 11-B.

11.7.1 Requirements for Class B.1

Buildings assigned to Class B.1 shall conform to all the requirements for Category A and to the additional requirements for Category B in other chapters of these provisions exclusive of the requirements for Class B.2.

11.7.1 Requirements for Class B.2

Buildings assigned to Class B.2 shall conform to all the requirements for Class B.1 and to the additional requirements of this section.

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

B. Discontinuous Members. Columns supporting reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement at the spacing s_o as defined in Sec. A.9.5.1 of Ref. 11.1 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as defined in Sec. 11.1.1.G of this chapter.

11.8 SEISMIC PERFORMANCE CATEGORIES C AND D

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

11.8.1 Moment Frames

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

11.8.2 Seismic Resisting System

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

11.8.3 Frame Components Not Part of the Seismic Resisting System

All frame components assumed not to be part of the Seismic Resisting System shall conform to Sec. 3.3.4(C) of these provisions and to Sec. A.8 of Ref. 11.1.

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

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

[a]Values shown are for minimum concrete compressive strength of 3,000 psi at 28 days. Values are for natural stone aggregate concrete and bolts of at least A-307 quality. Bolts shall have a standard bolt head or equal deformity in the embedded portion. Values are based upon a bolt spacing of 12 diameters with a minimum edge distance of 6 diameters. Such spacing and edge distance may be reduced 50 percent with an equal reduction in value. Use linear interpolation for intermediate spacings and edge margins.

[b]A minimum embedment of 9 bolt diameters shall be provided for anchor bolts located in the top of columns for buildings located in Seismicity Index Areas 3 and 4.

TABLE 11-B Definition of Class B.1 and B.2 of Seismic Performance Category B

Map Area Number[a]	Coefficient A_v[b]	Seismic Hazard Exposure Group	Class
2	$0.05 \leq A_v < 0.10$	I or II	B.1
2	$0.05 \leq A_v < 0.10$	III	B.2
3 or 4	$0.10 \leq A_v < 0.15$	All	B.2

[a]From Figure 1-2.

[b]From Figure 1-4.

Chapter 12

MASONRY

Replaced by TC 5-17/1A-85 & TC 17/1E-85

12.1 REFERENCE DOCUMENTS

The quality and testing of masonry and steel in, and the design and construction of, masonry 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. 12.1 American Standard Building Code Requirements for Masonry, A41.1-1953 (R 1970), American National Standards Institute.

Ref. 12.2 Building Code Requirements for Concrete Masonry Structures, ACI 531-79 (Revised 1983), American Concrete Institute.

Ref. 12.3 Building Code Requirements for Engineered Brick Masonry (1969), Brick Institute of America.

Ref. 12.4 Building Code Requirements for Reinforced Masonry, A 41.2-1960 (R 1970), American National Standards Institute.

Ref. 12.5 Specification for the Design and Construction of Load Bearing Concrete Masonry (1970), National Concrete Masonry Association.

Ref. 12.6 Uniform Building Code (1985 Edition), Chapter 24: Masonry, International Conference of Building Officials.

12.1.1 Modifications to Reference 12.6

Reference 12.6 shall be modified as follows:

A. Modifications to Section 2402. The following referenced Material Standards and Test Methods shall be used in place of the standards referenced in Section 2402(b) and 2402(c).

1. Masonry Units - Clay or Shale

   a. Standard Specification for Structural Clay Load-Bearing Wall Tile, ASTM C 34-62 (1975)

b. Standard Specification for Structural Clay Non-Load-Bearing Tile, ASTM C 56-71 (1981)

c. Standard Specification for Building Brick (Solid Masonry Units Made from Clay or Shale), ASTM C62-81

d. Standard Specifications for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units, ASTM C 126-82

e. Standard Specification for Facing Brick (Solid Masonry Units Made from Clay or Shale), ASTM C 216-81

f. Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale), ASTM C652-81a

g. Standard Specification for Structural Clay Facing Tile, ASTM C 212-60 (1981)

2. Masonry Units - Concrete

   a. Standard Specification for Concrete Building Brick, ASTM C 55-75 (1980)

   b. Standard Specification for Hollow Load-Bearing Concrete Masonry Units, ASTM C 90-75 (1981)

   c. Standard Specification for Non-Load-Bearing Concrete Masonry Units, ASTM C 129-75 (1980)

   d. Standard Specification for Solid Load-Bearing Concrete Masonry Units, ASTM C 145-75(1981)

3. Masonry Units - Other

   a. Standard Specification for Calcium Silicate Face Brick (Sand-Lime Brick), ASTM C 73-75

   b. Glass Block

      i. Glass block may be solid or hollow and contain inserts.

      ii. All mortar contact surfaces shall be treated to ensure adhesion between mortar and glass.

   c. Unburned Clay Masonry Units

   d. Cast Stone

   e. Reclaimed Units

      i. Reclaimed or previously used masonry units shall meet the applicable requirements as for new masonry units of the same material for their intended use.

4. Metal Ties and Anchors

   a. Metal ties and anchors shall be made of a material having a minimum tensile yield stress of 30,000 psi.

   b. All such items not fully embedded in mortar or grout shall be coated with copper, cadmium, zinc or a metal having at least equivalent corrosion-resistant properties.

5. Mortar

   a. Standard Specification for Mortar for Unit Masonry, ASTM C 270-82

6. Grout

   a. Standard Specification for Grout for Reinforced and Nonreinforced Masonry, ASTM C 476-80

7. Reinforcement

   a. Standard Specification for Deformed Steel Wire for Concrete Reinforcement, ASTM A 496-78

   b. Standard Specification for Deformed and Plain Billet-Steel Bars Concrete Reinforcement, ASTM A 615-82

   c. Standard Specification for Rail-Steel Deformed and Plain Bars for Concrete Reinforcement, ASTM A 616-82a

   d. Standard Specification for Axle-Steel Deformed and Plain Bars for Concrete Reinforcement, ASTM A 617-82a

   e. Standard Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement, ASTM A 706-82a

8. Test Methods

   a. Standard Methods of Sampling and Testing Brick and Structural Clay Tile, ASTM C 67-83

   b. Standard Methods of Sampling and Testing Concrete Masonry Units, ASTM C 140-75 (1980)

   c. Standard Methods for Field Testing of Grout, ASTM C 1019-84

   d. Standard Methods for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry, ASTM C 780-80

e. Tests of Masonry - See Section 2405(c)

B. Modification to Section 2405(c)2. Replace the first sentence of the sixth paragraph with the following:

"Prisms shall be capped and tested in accordance with the relevant portions of ASTM C 140 for prisms of concrete masonry units and of ASTM C 67 for prisms of clay or shale masonry units."

C. Modifications to Section 2407. Replace Section 2407(h)1 with the following:

"2407(h)1. General. Masonry structures shall be designed in accordance with the design requirements of this chapter and the special provisions for each seismic performance category given in this section."

Replace Section 2407(h)2 with the following:

"2407(h)2. Special Provisions for Seismic Performance Category A and Class B.1 of Seismic Performance Category B. There are no special design and construction provisions in this section for structures built in Seismic Performance Category A or Class B.1 of Category B."

Replace the title and first sentence of Section 2407(h)3 with the following:

"2407(h)3. Special Provisions for Class B.2 of Seismic Performance Category B. Masonry structures in Class B.2 of Seismic Performance Category B shall comply with the following special provisions:"

Replace the title and first sentence of Section 2407(h)4 with the following:

"2407(h)4. Special Provisions for Seismic Performance Category C. Masonry structures built in Seismic Performance Category C shall be designed and constructed in accordance with the requirements for Seismic Performance Category B and with the following additional requirements.

Replace the phrase "seismic Zones No. 3 and No. 4" in Section 2407(h)4.G with the following phrase "Seismic Performance Category C and D."

Delete the phrase "specified in Chapter 23" from Section 2407(i)5.B.

D. Modifications to Section 2409(e)6. Delete the following sentence:

"When lapped bars are from 0 to 3 inches apart, the lap length shall be increased by 1.3 times."

Modify Equations 9-11 and 9-12 of Ref. 12.6 by replacing f_s with F_s.

E. <u>Modifications to Section 2411</u>. Replace Section 2411(b)3.A with the following:

> "2411(b)3.A. Loads and load combinations shall be in accordance with Chapter 3, Structural Design Requirements.

12.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, ϕ, and 2.5 times the allowable working stress determined from one of the reference documents listed in Sec. 12.1 including the modifications 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 the masonry	$\phi = 0.4$
When permitted to consider masonry tension perpendicular to the bed joints, i.e., vertically in normal construction	$\phi = 0.4$
When considering splice length of reinforcement	$\phi = 0.4$

12.3 RESPONSE MODIFICATION COEFFICIENTS

The R factors of Table 3-B for reinforced masonry shall apply, provided masonry is designed in accordance with Ref. 12.6, Section 2409, and the requirements of the following sections of Ref. 12.6 are met:

1. 2407(h)4.B
2. 2407(h)4.C
3. 2407(h)4.D

The R factors of Table 3-B for unreinforced masonry shall apply for all other masonry.

12.4 SEISMIC PERFORMANCE CATEGORY A

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

12.5 SEISMIC PERFORMANCE CATEGORY B

For masonry construction, Seismic Performance Category B is divided into two classes, B.1 and B.2, as defined in Table 11-B.

12.5.1 Requirements for Class B.1

Buildings assigned to Class B.1 shall conform to all the requirements in other chapters of these provisions for Category A and to the additional requirements for Category B exclusive of the requirements for Class B.2. Buildings higher than 50 feet shall conform to the requirements of Class B.2 unless approved by the Regulatory Agency based upon substantiating data.[5]

12.5.2 Requirements for Class B.2

Buildings assigned to Class B.2 shall conform to the requirements of Ref. 12.6 and to the additional requirements of this section.

12.5.3 Construction Limitations

A. Screen Walls. All screen walls shall be reinforced in accordance with this section. Joint reinforcement shall be considered effective in resisting stresses. The units of a panel shall be so arranged that either the horizontal or the vertical joint containing reinforcing is continuous without offset. This continuous joint shall be reinforced with a minimum steel area of 0.03 square inch. Reinforcement shall be embedded in mortar or grout.

Joint reinforcement may be composed of two wires made with welded ladder or trussed wire cross ties. In calculating the resisting capacity of the system, compression and tension in the spaced wires may be utilized. Ladder wire reinforcement shall not be spliced and shall be the widest that the mortar joint will accommodate allowing 1/2 inch of mortar cover.

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[5]The height limit is arbitrary and should be reviewed for need and numerical value.

Each panel shall be supported on all edges by a structural member of concrete, masonry, or steel.

B. Construction Type. Wythes of cavity walls of structural masonry shall be designed and reinforced as if independent walls.

12.5.4 Material Limitations

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

Unburned Clay Masonry

Structural Clay Load-bearing Wall Tile (ASTM C 34)

Masonry Cement (mortar with air content greater than 15 percent)

12.6 SEISMIC PERFORMANCE CATEGORY C

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

12.6.1 Construction Limitations

Masonry components shall be constructed to conform to the limitations of this section.

A. Tie Anchorages. In addition to the requirements of Ref. 12.6 Section 2409(b)5.B. for tie anchorages, an extension of at least 6 tie diameters but not less than 4 inches at the free end of the tie shall be provided.

B. Stacked Bond Construction. The minimum ratio of horizontal reinforcement shall be 0.0015 for all structural walls of stacked bond construction. The maximum spacing of horizontal reinforcement shall not exceed 24 inches. Where reinforced hollow unit construction forms part of the seismic resisting system, the construction shall be grouted solid and all head joints shall be made solid through the use of open end units. The opening formed by the open end units shall satisfy the requirements of Sec. 12.6.1.C.

C. Hollow Unit Masonry. Vertical cells to be filled shall have vertical alignment sufficient to maintain a clear, unobstructed continuous vertical cell measuring not less than 2 inches by 3 inches. If walls are battered or if alignment is offset, the 2-inch by 3-inch clear opening shall be maintained as measured from course to course.

D. Shrinkage of Concrete Units. If the concrete masonry units used for structural purposes have a linear shrinkage exceeding 0.065 percent from the saturated to the oven-dry condition, special consideration shall be given to design for shrinkage effects.

12.6.2 Shear Wall Requirements

Shear walls shall comply with the requirements of this section.

A. Reinforcement. The minimum ratio of reinforcement for shear walls shall be 0.0015 in each direction. The maximum spacing of reinforcement in each direction shall be the smaller of the following dimensions: one-third the length and height of the element but not more than 48 inches. The area and spacing of reinforcement perpendicular to the shear reinforcement shall be at least equal to that of the required shear reinforcement. The portion of the reinforcement required to resist shear shall be uniformly distributed.

EXCEPTION:

For shear walls constructed using running bond, the ratio of reinforcement may be decreased to 0.0007 provided that all shear is resisted by the reinforcement. The sum of the ratios of horizontal and vertical reinforcement shall not be less than 0.002.

Reinforcement required to resist wall shear shall be terminated with a standard hook that terminates beyond the boundary reinforcing at the end of the wall sections. The hook may be turned up, down, or horizontally and shall be embedded in mortar or grout. Wall reinforcement terminating in boundary columns of beams shall be fully anchored into the boundary elements.

B. Boundary Members. Where cross walls or boundary members form a part of the shear wall system, the intersection shall be constructed as required for the walls themselves. Connections to concrete shall conform to Ref. 12.6, Sec. 2407(h)4.H. Where the boundary members are of structural steel, the shear transfer between the wall and the boundary members shall be developed by fully encasing the element in grout, by dowels, bolts, or shear lugs, or by similar approved methods.

When the structural system, as described in Chapter 3 and Table 3-B, consists of substantially complete vertical load-carrying frame, boundary members shall be provided at each end of the wall. The members shall be of the same construction as the frame columns. Where the frame is a special moment frame, those columns shall conform to the requirements for such members in Chapters 10 and 11.

The required vertical boundary members and such other similar vertical elements as may be required shall be designed to carry all the vertical forces resulting from the wall loads, the tributary dead and live loads, and the seismic forces prescribed in these provisions.

Horizontal reinforcing in the walls shall be anchored to the vertical elements. Where the boundary element is structural steel, this shall be accomplished by welding or by extension, with bends if required, into grout fully surrounding the column.

C. Stresses. For load combinations including in-plane seismic forces, allowable compression stresses at any point shall not exceed those allowed for axial compression.

Vertical stresses in shear walls shall be determined from the combined effects of vertical load and from the overturning effects of lateral loads. Minimum vertical loads shall be considered. Eq. 3-2a shall be used for unreinforced masonry design.

In computing the shear resistance of the wall, only the web shall be considered.

EXCEPTION:

For pier type wall elements that do not extend from floor to floor, compression stresses under combined loading at any point may be limited to those allowed for flexural compression provided that Ref. 12.6, Section 2406(e)1, Equation (6-21), is also satisfied.

D. Horizontal Components. When shear reinforcing is required for loads that include seismic effects and diagonal bars are not provided, reinforcement approximately perpendicular to the required shear reinforcement shall be provided equal in amount and spaced not further apart than is required for the shear reinforcing. Horizontal reinforcing shall anchor into or be continuous through the pier elements. Horizontal components may be separated from the shear wall system by means of joints. The joints shall provide for building movement determined in accordance with Sec. 3.8. The horizontal components shall be anchored to the building and designed as otherwise required by these provisions.

12.6.3 Material Limitations

The following materials shall not be used:

Unburned Clay Masonry

Structural Clay Load-bearing Wall Tile, ASTM C 34

Structural Clay Nonload-bearing Wall Tile, ASTM C 56

12.7 SEISMIC PERFORMANCE CATEGORY D

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

12.7.1 Construction Limitations

Materials for mortar and grout for structural masonry shall be measured in suitable calibrated devices. Shovel measurements are not acceptable. An approved admixture of a type that reduces early water loss and produces a net expansion action shall be used for grout for structural masonry unless it can be demonstrated that shrinkage cracks will not develop in the grout. The thickness of the grout between masonry units and reinforcing shall be a minimum of 1/2 inch for structural masonry.

A. Minimum Grout Space for Grouted Masonry. The minimum grout space for structural reinforced grouted masonry shall be 2-1/2 inches for low-lift construction and 3-1/2 inches for high-lift construction.

B. Reinforced Hollow Unit Masonry. Structural reinforced hollow unit masonry shall conform to requirements below:

1. Wythes and elements shall be at least 8 inches in nominal thickness with clear, unobstructed continuous vertical cells, without offsets, large enough to enclose a circle of at least 3-1/2 inches in diameter and with a minimum area of 15 square inches.

2. All grout shall be coarse grout. Grout consolidation shall be by mechanical vibration only. All grout shall be reconsolidated after excess moisture has been absorbed but before workability has been lost.

3. Vertical reinforcement shall be securely held in position at tops, bottoms, splices, and at intervals not exceeding 112 bar diameters. Approved intermediate centering clips of caging devices shall be used in high-lift construction, as required, to hold the vertical bars. Horizontal wall reinforcement shall be securely tied to the vertical reinforcement or held in place during grouting by equivalent means.

4. In wythes of less than 10 inch nominal thickness, in any vertical cell, there shall be a maximum of one No. 10 bar or two No. 8 bars with splices staggered for the two-bar situation.

5. The minimum nominal column dimension shall be 12 inches. The exception permitted under Ref. 12.6, Section 2407(h)4.E(ii) shall not apply.

C. Stacked Bond Construction. All stacked bond construction shall conform to the following requirements:

1. The minimum ratio of horizontal reinforcement shall be 0.0015 for nonstructural masonry and 0.0025 for structural masonry. The maximum spacing of horizontal reinforcing shall not exceed 24 inches for nonstructural masonry or 16 inches for structural masonry.

2. Reinforced hollow unit construction that is part of the

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

3. Other reinforced hollow unit construction used structurally, but not part of the seismic resisting system, shall be grouted solid and all head joints shall be made solid by the use of open end units.

12.7.2 Special Inspection

Special inspection shall be provided for all structural masonry.

Appendix to Chapter 12

ANCHOR BOLTS IN MASONRY

INTRODUCTION

The BSSC Technical Committee on Masonry unanimously agreed that there is an inadequate treatment of the design requirements for anchor bolts in the provisions. The tentative provisions presented in this appendix are based on the available research from 1934 to 1980 and on research conducted at Clemson University in 1983. They should be used carefully and with consideration of subsequent research findings.

SHEAR AND TENSION ON EMBEDDED ANCHOR BOLTS

Allowable loads and placement requirements for plate anchor bolts, headed anchor bolts, and bent bar anchor bolts shall be determined in accordance with Sections 2406(h)1., 2., 3., 4., and 5 of Ref. 12.6.

Notations used in this section shall be defined as follows:

- B_t = Allowable tension force on anchor bolts (lb)
- B_v = Allowable shear force on anchor bolts (lb)
- b_t = Computed tension force on anchor bolts (lb)
- b_v = Computed shear force on anchor bolts (lb)
- A_p = Area tension (pull out) cone projected onto the surface of masonry (in.^{2})
- A_b = Cross-sectional area of anchor bolts (in.^{2})
- ℓ_b = Embedment depth of anchor bolts (in.)
- ℓ_{be} = Edge distance, the least length measured from the edge of masonry to the surface of the anchor bolt (in.)

Tension

Allowable loads in tension shall be the lesser of Eq. 10A-1 or 10A-2:

$$B_t = 0.5 A_p \sqrt{f'_m} \qquad (10A\text{-}1)$$

or

$$B_t = 0.2 A_b f_y. \qquad (10A\text{-}2)$$

The area Ap shall be the lesser of Eq. 10A-3 or 10A-4, and where the projected areas of adjacent anchor bolts overlap, Ap of each bolt shall be reduced by one-half of the overlapping area:

$$A_p = \pi \ell_b^2 \qquad (10A\text{-}3)$$

or

$$A_p = \pi \ell_{be}^2. \qquad (10A\text{-}4)$$

The effective embedment length, ℓ_b for plate or headed anchor bolts 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, and ℓ_b for bent bar anchors perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter.

Shear

Allowable loads in shear, where ℓ_{be} equals or exceeds 12 bolt diameters shall be the lesser of Eq. 10A-5 or 10A-6:

$$B_v = 350 \sqrt[4]{f'_m A_b} \qquad (10A\text{-}5)$$

or

$$B_v = 0.12 A_b f_y. \qquad (10A\text{-}6)$$

Where ℓ_{be} is less than 12 bolt diameters, the value of B_v shall be reduced by linear interpolation to zero at an ℓ_{be} distance of one (1) inch.

Combined Shear and Tension

Anchor bolts subjected to combined shear and tension shall be designed in accordance with Eq. 10A-7:

$$b_t/B_t + b_v/B_v \leq 1.0 \qquad (10A\text{-}7)$$

Minimum Edge Distance

The minimum value of ℓ_{be} measured from the edge of the masonry to the surface of the anchor bolt shall be one inch.

Minimum Embedment Depth

The minimum embedment depth shall be four bolt diameters.

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