

PREFACE

Introduction

The devastating earthquake of October 08, 2005 made it abundantly clear that earthquake provisions of the Pakistan Building Code 1986 need to be comprehensively bolstered so that safety for all communities are ensured. This has been encapsulated in these “Seismic Provisions”. The objective of these provisions is to establish minimum regulations for earthquake considerations in building systems.

These Seismic Provisions in the Pakistan Building Code are founded on broad-based principles that make the use of new materials and new construction systems possible.

The Seismic Provisions are compatible with the Uniform Building Code 1997 (of USA), the American Concrete Institute ACI 318-05, American Institute of Steel Construction ANSI/AISC 341-05, American Society of Civil Engineers SEI/ASCE 7-05 and ANSI/ASCE 7-93. Revisions to these provisions will be made every five years or as and when deemed necessary, which will allow updating the provisions of this code continually.

Development

The Ministry of Housing & Works (MOHW) Government of Pakistan (GoP) assigned the task of developing the Seismic Provisions to the National Engineering Services Pakistan (Pvt.) Limited (NESPAK). NESPAK submitted various drafts for scrutiny to an Experts Committee formed by the MOHW. NESPAK worked in close collaboration with International Code Council (ICC), USA. The final draft was sent to the Pakistan Engineering Council (PEC) for vetting. PEC formed a “Core Group” of individuals drawn from across the country, representing various stakeholders. It was this Core Group that held intimate deliberations with experts from NESPAK and gave final shape to the document.

While these code provisions protect public safety, it has been ensured that these do not unnecessarily increase costs or restrict the use of new materials and technology.

Maintenance

The Seismic Provisions of the code shall be kept updated with the revisions suggested by a standing committee working under the aegis of the PEC. This committee will hold discussions with representatives from industry, engineering professionals and other stakeholders in an open code development process before any change is suggested.

Waiver

While utmost care was taken by members that contributed in developing these provisions of the code, the individuals and their organizations accept no liability resulting from the compliance or noncompliance by practitioners. Properly licensed and experienced engineers and construction professionals should be engaged to interpret the use of these provisions. The power to ensure compliance vests only with the Government of Pakistan.

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A document as important and detailed as code has input from many individuals. While it is not possible to acknowledge everyone's effort here, it is important to list those whose input was critical.

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SOURCE DOCUMENTS

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- IAEE (1986), *Guidelines for earthquake resistant non-engineered construction*, International Association for Earthquake Engineering, Japan.

g) CHAPTER 10 Architectural Elements

- ASCE (1993), *Minimum Design Loads for Building and Other Structures*, ANSI/ASCE 7-93, American Society of Civil Engineers, Reston, VA.
- ASCE (2005), *Minimum Design Loads for Building and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA.

h) CHAPTER 11 Mechanical and Electrical Systems

- ASCE (1993), *Minimum Design Loads for Building and Other Structures*, ANSI/ASCE 7-93, American Society of Civil Engineers, Reston, VA.
- ASCE (2005), *Minimum Design Loads for Building and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA.

TABLE OF CONTENTS

PREFACE	i
ACKNOWLEDGEMENTS	ii
SOURCE DOCUMENTS	iv
CHAPTER 1: SCOPE	
1.1 Objective and General Principles	1-1
1.2 Scope	1-1
CHAPTER 2: SEISMIC HAZARD	
2.1 Scope	2-1
2.2 Design Basis Ground Motion	2-1
2.3 Seismic Zones	2-1
2.4 Site-specific Hazard Analysis	2-1
2.5 Modeling of Ground Motion	2-1
CHAPTER 3: SITE CONSIDERATIONS	
3.1 Scope	3-1
3.2 Potential Fault Rupture Hazard	3-1
3.3 Potential Liquefaction	3-1
3.4 Potential Landslide and Slope Instability	3-1
3.5 Sensitive Clays	3-1
CHAPTER 4: SOILS AND FOUNDATIONS	
4.1 Symbols and Notations	4-1
4.2 Scope	4-1
4.3 Determination of Soil Conditions	4-1
4.3.1 Site Geology and Soil Characteristics	4-1
4.4 Soil Profile Types	4-1
4.4.1 Scope	4-1
4.4.2 Definitions	4-1
4.5 Foundations Construction in Seismic Zones 3 and 4	4-3
4.5.1 General	4-3
4.5.2 Soil Capacity	4-4
4.5.3 Superstructure-to-Foundation Connection	4-4
4.5.4 Foundation-Soil Interface	4-4
4.5.5 Special Requirements for Piles and Caissons	4-4
CHAPTER 5: STRUCTURAL DESIGN REQUIREMENTS	
Division I General Design Requirements	
5.1 Symbols and Notations	5-1
5.2 Scope	5-1
5.3 Definitions	5-1
5.4 Standards	5-2
5.5 Design	5-2

5.5.1	General	5-2
5.5.2	Rationality	5-2
5.5.3	Erection of Structural Framing	5-3
5.6	Dead Loads	5-3
5.6.1	General	5-3
5.6.2	Partition Loads	5-3
5.7	Live Loads	5-3
5.7.1	General	5-3
5.7.2	Critical Distribution of Live Loads	5-4
5.7.3	Floor Live Loads	5-4
5.7.4	Roof Live Loads	5-4
5.7.5	Reduction of Live Loads	5-5
5.7.6	Alternate Floor Live Load Reduction	5-6
5.8	Snow Loads	5-6
5.9	Wind Loads	5-6
5.10	Earthquake Loads	5-6
5.11	Other Minimum Loads	5-7
5.11.1	General	5-7
5.11.2	Other Loads	5-7
5.11.3	Impact Loads	5-7
5.11.4	Anchorage of Concrete and Masonry Walls	5-7
5.11.5	Interior Wall Loads	5-7
5.11.6	Retaining Walls	5-7
5.11.7	Water Accumulation	5-7
5.11.8	Hydrostatic Uplift	5-8
5.11.9	Flood-resistant Construction	5-8
5.11.10	Heliport and Helistop Landing Areas	5-8
5.11.11	Prefabricated Construction	5-8
5.12	Combinations of Loads	5-8
5.12.1	General	5-8
5.12.2	Load Combinations Using Strength Design or Load and Resistance Factor Design	5-9
5.12.3	Load Combinations Using Allowable Stress Design	5-9
5.12.4	Special Seismic Load Combinations	5-10
5.13	Deflection	5-10

Division II Snow Loads

5.14	Snow Loads	5-11
------	------------	------

Division III Wind Design

5.15	Symbols and Notations	5-12
5.16	General	5-12
5.17	Definitions	5-12
5.18	Basic Wind Speed	5-13
5.19	Exposure	5-13
5.20	Design Wind Pressures	5-13
5.21	Primary Frames and Systems	5-13
5.21.1	General	5-13
5.21.2	Method 1 (Normal Force Method)	5-13
5.21.3	Method 2 (Projected Area Method)	5-14
5.22	Elements and Components of Structures	5-14
5.23	Open Frame Towers	5-14
5.24	Miscellaneous Structures	5-14

5.25	Occupancy Categories	5-14
Division IV Earthquake Design		
5.26	Symbols and Notations	5-15
5.27	General	5-16
5.27.1	Purpose	5-16
5.27.2	Minimum Seismic Design	5-16
5.27.3	Seismic and Wind Design	5-16
5.28	Definitions	5-17
5.29	Criteria Selection	5-20
5.29.1	Basis for Design	5-20
5.29.2	Occupancy Categories	5-20
5.29.3	Site Geology and Soil Characteristics	5-20
5.29.4	Site Seismic Hazard Characteristics	5-20
5.29.5	Configuration Requirements	5-21
5.29.6	Structural Systems	5-21
5.29.7	Height Limits	5-22
5.29.8	Selection of Lateral-force Procedure	5-22
5.29.9	System Limitations	5-23
5.29.10	Alternative Procedures	5-23
5.30	Minimum Design Lateral Forces and Related Effects	5-24
5.30.1	Earthquake Loads and Modeling Requirements	5-24
5.30.2	Static Force Procedure	5-26
5.30.3	Determination of Seismic Factors	5-28
5.30.4	Combinations of Structural Systems	5-28
5.30.5	Vertical Distribution of Force	5-29
5.30.6	Horizontal Distribution of Shear	5-29
5.30.7	Horizontal Torsional Moments	5-30
5.30.8	Overturning	5-30
5.30.9	Drift	5-31
5.30.10	Storey Drift Limitation	5-31
5.30.11	Vertical Component	5-32
5.31	Dynamic Analysis Procedures	5-32
5.31.1	General	5-32
5.31.2	Ground Motion	5-32
5.31.3	Mathematical Model	5-33
5.31.4	Description of Analysis Procedures	5-33
5.31.5	Response Spectrum Analysis	5-33
5.31.6	Time-history Analysis	5-34
5.32	Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures	5-35
5.32.1	General	5-35
5.32.2	Design for Total Lateral Force	5-36
5.32.3	Specifying Lateral Forces	5-37
5.32.4	Relative Motion of Equipment Attachments	5-37
5.32.5	Alternative Designs	5-37
5.33	Detailed Systems Design Requirements	5-37
5.33.1	General	5-37
5.33.2	Structural Framing Systems	5-38
5.34	Nonbuilding Structures	5-42
5.34.1	General	5-42
5.34.2	Lateral Force	5-42
5.34.3	Rigid Structures	5-43
5.34.4	Tanks with Supported Bottoms	5-43

5.34.5	Other Nonbuilding Structures	5-43
5.35	Earthquake-Recording Instrumentations	5-44
5.35.1	General	5-44
5.35.2	Location	5-44
5.35.3	Maintenance	5-44
5.35.4	Instrumentation of Existing Buildings	5-44

CHAPTER 6: STRUCTURAL TESTS AND INSPECTIONS

6.1	Special Inspections	6-1
6.1.1	General	6-1
6.1.2	Special Inspector	6-1
6.1.3	Duties and Responsibilities of the Special Inspector	6-1
6.1.4	Standards of Quality	6-1
6.1.5	Types of Work	6-2
6.1.6	Continuous and Periodic Special Inspection	6-5
6.1.7	Approved Fabricators	6-5
6.2	Structural Observation	6-5
6.3	Nondestructive Testing	6-6
6.4	Prefabricated Construction	6-7
6.4.1	General	6-7
6.4.2	Tests of Materials	6-7
6.4.3	Tests of Assemblies	6-7
6.4.4	Connections	6-7
6.4.5	Pipes and Conduits	6-7
6.4.6	Certificate and Inspection	6-8

CHAPTER 7: STRUCTURAL CONCRETE

7.1	Symbols and Notations	7-1
7.2	Definitions	7-3
7.3	General Requirements	7-5
7.3.1	Scope	7-5
7.3.2	Analysis and Proportioning of Structural Members	7-5
7.3.3	Strength Reduction Factors	7-6
7.3.4	Concrete in Members Resisting Earthquake Induced Forces	7-7
7.3.5	Reinforcement in Members Resisting Earthquake Induced Forces	7-7
7.3.6	Welded Splices	7-7
7.3.7	Anchoring to Concrete	7-7
7.4	Flexural Members of Special Moment Frames	7-7
7.4.1	Scope	7-7
7.4.2	Longitudinal Reinforcement	7-8
7.4.3	Transverse Reinforcement	7-8
7.4.4	Shear Strength Requirements	7-9
7.5	Special Moment Frame Members Subjected to Bending and Axial Load	7-9
7.5.1	Scope	7-9
7.5.2	Minimum Flexural Strength of Columns	7-9
7.5.3	Longitudinal Reinforcement	7-10
7.5.4	Transverse Reinforcement	7-10
7.5.5	Shear Strength Requirements	7-12
7.6	Joints of Special Moment Frames	7-12
7.6.1	General Requirements	7-12
7.6.2	Transverse Reinforcement	7-12

7.6.3	Shear Strength	7-13
7.6.4	Development Length of Bars in Tension	7-13
7.7	Special Moment Frames Constructed using Precast Concrete	7-14
7.8	Special Reinforced Concrete Structural Walls and Coupling Beams	7-15
7.8.1	Scope	7-15
7.8.2	Reinforcement	7-15
7.8.3	Design Forces	7-15
7.8.4	Shear Strength	7-15
7.8.5	Design for Flexure and Axial Loads	7-16
7.8.6	Boundary Elements of Special Reinforced Concrete Structural Walls	7-16
7.8.7	Coupling Beams	7-17
7.8.8	Construction Joints	7-18
7.8.9	Discontinuous Walls	7-18
7.9	Special Structural Walls Constructed using Precast Concrete	7-18
7.10	Structural Diaphragms and Trusses	7-18
7.10.1	Scope	7-18
7.10.2	Cast-in-place Composite-topping Slab Diaphragms	7-18
7.10.3	Cast-in-place Topping Slab Diaphragms	7-19
7.10.4	Minimum Thickness of Diaphragms	7-19
7.10.5	Reinforcement	7-19
7.10.6	Design Forces	7-19
7.10.7	Shear Strength	7-19
7.10.8	Boundary Elements of Structural Diaphragms	7-20
7.11	Foundations	7-20
7.11.1	Scope	7-20
7.11.2	Footings, Foundation Mats and Pile Caps	7-20
7.11.3	Grade Beams and Slabs on Grade	7-21
7.11.4	Piles, Piers and Caissons	7-21
7.12	Members not Designated as Part of the Lateral-force-resisting System	7-22
7.13	Requirements for Intermediate Moment Frames	7-23
7.14	Intermediate Precast Structural Walls	7-24

CHAPTER 8: STRUCTURAL STEEL

8.1	Symbols & Notations	8-1
-----	---------------------	-----

Division I Structural Steel Buildings

8.2	Definitions	8-5
8.3	Scope	8-9
8.4	Loads, Load Combinations and Nominal Strengths	8-9
8.4.1	Loads and Load Combinations	8-9
8.4.2	Nominal Strength	8-9
8.5	Structural Design Drawings and Specifications, Shop Drawings and Erection Drawings	8-10
8.5.1	Structural Design Drawings and Specifications	8-10
8.5.2	Shop Drawings	8-10
8.5.3	Erection Drawings	8-10
8.6	Materials	8-11
8.6.1	Material Specifications	8-11
8.6.2	Material Properties for Determination of Required Strength of Members and Connections	8-11
8.6.3	Heavy Section CVN Requirements	8-11

8.7	Connections, Joints and Fasteners	8-12
8.7.1	Scope	8-12
8.7.2	Bolted Joints	8-12
8.7.3	Welded Joints	8-12
8.7.4	Protected Zone	8-13
8.7.5	Continuity Plates and Stiffeners	8-13
8.8	Members	8-14
8.8.1	Scope	8-14
8.8.2	Classification of Sections for Local Buckling	8-14
8.8.3	Column Strength	8-14
8.8.4	Column Splices	8-15
8.8.5	Column Bases	8-15
8.8.6	H-Piles	8-16
8.9	Special Moment Frames (SMF)	8-17
8.9.1	Scope	8-17
8.9.2	Beam-to-Column Connections	8-17
8.9.3	Panel Zone of Beam-to-column Connections (beam web parallel to column web)	8-18
8.9.4	Beam and Column Limitations	8-19
8.9.5	Continuity Plates	8-19
8.9.6	Column-Beam Moment Ratio	8-20
8.9.7	Lateral Bracing at Beam-to-Column Connections	8-21
8.9.8	Lateral Bracing of Beams	8-22
8.9.9	Column Splices	8-22
8.10	Intermediate Moment Frames (IMF)	8-22
8.10.1	Scope	8-22
8.10.2	Beam-to-Column Connections	8-23
8.10.3	Panel Zone of Beam-to-Column Connections (beam web parallel to column web)	8-23
8.10.4	Beam and Column Limitations	8-23
8.10.5	Continuity Plates	8-24
8.10.6	Column-Beam Moment Ratio	8-24
8.10.7	Lateral Bracing at Beam-to-Column Connections	8-24
8.10.8	Lateral Bracing of Beams	8-24
8.10.9	Column Splices	8-24
8.11	Ordinary Moment Frames (OMF)	8-24
8.11.1	Scope	8-24
8.11.2	Beam-to-Column Connections	8-25
8.11.3	Panel Zone of Beam-to-Column Connections (beam web parallel to column web)	8-26
8.11.4	Beam and Column Limitations	8-26
8.11.5	Continuity Plates	8-26
8.11.6	Column-Beam Moment Ratio	8-27
8.11.7	Lateral Bracing at Beam-to-Column Connections	8-27
8.11.8	Lateral Bracing of Beams	8-27
8.11.9	Column Splices	8-27
8.12	Special Truss Moment Frames (STMF)	8-27
8.12.1	Scope	8-27
8.12.2	Special Segment	8-27
8.12.3	Strength of Special Segment Members	8-28
8.12.4	Strength of Non-Special Segment Members	8-28
8.12.5	Width-Thickness Limitations	8-29
8.12.6	Lateral Bracing	8-29
8.13	Special Concentrically Braced Frames (SCBF)	8-29

8.13.1	Scope	8-29
8.13.2	Members	8-29
8.13.3	Required Strength of Bracing Connections	8-30
8.13.4	Special Bracing Configuration Requirements	8-31
8.13.5	Column Splices	8-31
8.13.6	Protected Zone	8-31
8.14	Ordinary Concentrically Braced Frames (OCBF)	8-32
8.14.1	Scope	8-32
8.14.2	Bracing Members	8-32
8.14.3	Special Bracing Configuration Requirements	8-32
8.14.4	Bracing Connections	8-32
8.14.5	OCBF above Seismic Isolation Systems	8-33
8.15	Eccentrically Braced Frames (EBF)	8-33
8.15.1	Scope	8-33
8.15.2	Links	8-33
8.15.3	Link Stiffeners	8-35
8.15.4	Link-to-Column Connections	8-35
8.15.5	Lateral Bracing of Link	8-36
8.15.6	Diagonal Brace and Beam Outside of Link	8-36
8.15.7	Beam-to-Column Connections	8-37
8.15.8	Required Strength of Columns	8-37
8.15.9	Protected Zone	8-37
8.15.10	Demand Critical Welds	8-37
8.16	Buckling-Restrained Braced Frames (BRBF)	8-38
8.16.1	Scope	8-38
8.16.2	Bracing Members	8-38
8.16.3	Bracing Connections	8-39
8.16.4	Special Requirements Related to Bracing Configuration	8-40
8.16.5	Beams and Columns	8-40
8.16.6	Protected Zone	8-40
8.17	Special Plate Shear Walls (SPSW)	8-41
8.17.1	Scope	8-41
8.17.2	Webs	8-41
8.17.3	Connections of Webs to Boundary Elements	8-42
8.17.4	Horizontal and Vertical Boundary Elements	8-42
8.18	Quality Assurance Plan	8-43
8.18.1	Scope	8-43

Division II **Composite Structural Steel and Reinforced Concrete Buildings**

8.19	Definitions	8-44
8.20	Scope	8-45
8.21	General Seismic Design Requirements	8-46
8.22	Loads, Load Combinations and Nominal Strengths	8-46
8.22.1	Loads and Load Combinations	8-46
8.22.2	Nominal Strength	8-46
8.23	Materials	8-47
8.23.1	Structural Steel	8-47
8.23.2	Concrete and Steel Reinforcement	8-47
8.24	Composite Members	8-47
8.24.1	Scope	8-47
8.24.2	Composite Floor and Roof Slabs	8-47
8.24.3	Composite Beams	8-47
8.24.4	Encased Composite Columns	8-47

8.24.5	Filled Composite Columns	8-50
8.25	Composite Connections	8-51
8.25.1	Scope	8-51
8.25.2	General Requirements	8-51
8.25.3	Nominal Strength of Connections	8-51
8.26	Composite Partially Restrained (PR) Moment Frames (C-PRMF)	8-52
8.26.1	Scope	8-52
8.26.2	Columns	8-53
8.26.3	Composite Beams	8-53
8.26.4	Moment Connections	8-53
8.27	Composite Special Moment Frames (C-SMF)	8-53
8.27.1	Scope	8-53
8.27.2	Columns	8-53
8.27.3	Beams	8-53
8.27.4	Moment Connections	8-54
8.27.5	Column-Beam Moment Ratio	8-54
8.28	Composite Intermediate Moment Frames (C-IMF)	8-54
8.28.1	Scope	8-54
8.28.2	Columns	8-54
8.28.3	Beams	8-55
8.28.4	Moment Connections	8-55
8.29	Composite Ordinary Moment Frames (C-OMF)	8-55
8.29.1	Scope	8-55
8.29.2	Columns	8-55
8.29.3	Beams	8-55
8.29.4	Moment Connections	8-55
8.30	Composite Special Concentrically Braced Frames (C-CBF)	8-55
8.30.1	Scope	8-55
8.30.2	Columns	8-56
8.30.3	Beams	8-56
8.30.4	Braces	8-56
8.30.5	Connections	8-56
8.31	Composite Ordinary Braced Frames (C-OBF)	8-56
8.31.1	Scope	8-56
8.31.2	Columns	8-56
8.31.3	Beams	8-56
8.31.4	Braces	8-56
8.31.5	Connections	8-57
8.32	Composite Eccentrically Braced Frames (C-EBF)	8-57
8.32.1	Scope	8-57
8.32.2	Columns	8-57
8.32.3	Links	8-57
8.32.4	Braces	8-57
8.32.5	Connections	8-57
8.33	Ordinary Reinforced Concrete Shear Walls Composite with Structural Steel Elements (C-ORCW)	8-58
8.33.1	Scope	8-58
8.33.2	Boundary Members	8-58
8.33.3	Steel Coupling Beams	8-58
8.33.4	Encased Composite Coupling Beams	8-59
8.34	Special Reinforced Concrete Shear Walls Composite with Structural Steel Elements (C-SRCW)	8-59
8.34.1	Scope	8-59
8.34.2	Boundary Members	8-59
8.34.3	Steel Coupling Beams	8-59

8.34.4	Encased Composite Coupling Beams	8-60
8.35	Composite Steel Plate Shear Walls (C-SPW)	8-60
8.35.1	Scope	8-60
8.35.2	Wall Elements	8-60
8.35.3	Boundary Members	8-61
8.35.4	Openings	8-61
8.36	Structural Design Drawings and Specifications, Shop Drawings and Erection Drawings	8-61
8.37	Quality Assurance Plan	8-61

CHAPTER 9: MASONRY

9.1	Symbols and Notations	9-1
9.2	Scope	9-3
9.2.1	Design Methods	9-3
9.3	Definitions	9-4
9.4	Material Standards	9-6
9.4.1	Quality	9-6
9.4.2	Standards of Quality	9-6
9.4.3	Mortar and Grout	9-8
9.4.4	Mortar	9-8
9.4.5	Grout	9-8
9.4.6	Additives and Admixtures	9-9
9.4.7	Construction	9-9
9.4.8	Cold-weather Construction	9-9
9.4.9	Placing Masonry Units	9-10
9.4.10	Reinforcement Placing	9-11
9.4.11	Grouted Masonry	9-11
9.4.12	Aluminum Equipment	9-12
9.4.13	Joint Reinforcement	9-12
9.5	Quality Assurance	9-12
9.5.1	General	9-12
9.5.2	Scope	9-13
9.5.3	Compliance with f'_m	9-13
9.5.4	Mortar Testing	9-15
9.5.5	Grout Testing	9-15
9.6	General Design Requirements	9-15
9.6.1	General.	9-15
9.6.2	Working Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry	9-19
9.6.3	Working Stress Design and Strength Design Requirements for Reinforced Masonry	9-22
9.7	Working Stress Design of Masonry	9-24
9.7.1	General.	9-24
9.7.2	Design of Reinforced Masonry	9-26
9.7.3	Design of Unreinforced Masonry	9-32
9.8	Strength Design of Masonry	9-34
9.8.1	General	9-34
9.8.2	Reinforced Masonry	9-36
9.9	Empirical Design	9-49
9.9.1	Scope	9-49
9.9.2	Materials	9-51
9.9.3	Masonry with Seismic Bands	9-51
9.9.4	Confined Masonry	9-52
9.9.5	Reinforced Masonry	9-53

CHAPTER 10: ARCHITECTURAL ELEMENTS

10.1	Symbols and Notations	10-1
10.2	Seismic Loads Applied to Architectural Components	10-1
10.2.1	Component Force Application	10-1
10.2.2	Component Force Transfer	10-1
10.2.3	Architectural Component Deformation	10-1
10.2.4	Out-of-Plane Bending	10-2
10.3	Suspended Ceilings	10-2
10.3.1	Seismic Forces	10-2
10.3.2	Integral Construction	10-2
10.3.3	Access Floors	10-2
10.4	Partitions	10-3
10.4.1	General	10-3
10.4.2	Glass in Glazed Curtain Walls, Glazed Storefronts and Glazed Partitions	10-3

CHAPTER 11: MECHANICAL & ELECTRICAL SYSTEMS

11.1	Symbols and Notations	11-1
11.2	Seismic Loads applied to Mechanical and Electrical Components	11-1
11.2.1	Component Force Application	11-1
11.2.2	Component Force Transfer	11-2
11.2.3	Component Period	11-2
11.2.4	Component Attachment	11-2
11.3	Elevator Design Requirements	11-2

APPENDIX-A BACKGROUND FOR SEISMIC ZONING MAP

A.1	Symbols and Notations	A-1
A.2	Overview	A-1
A.2.1	General	A-1
A.2.2	Major Faults of Pakistan	A-1
A.2.3	Seismicity	A-2
A.3	Seismic Hazard Evaluation Procedure	A-3
A.3.1	PSHA Methodology	A-3
A.3.2	Source Modeling – Area and Fault Seismic Sources	A-4
A.3.3	Earthquake Recurrence Model	A-4
A.3.4	Maximum Magnitude	A-5
A.3.5	Attenuation Equations	A-5
A.3.6	Results of PSHA	A-6

CHAPTER 1

SCOPE

1.1 Objective and General Principles

The objective of the provisions described in this code is to prescribe the minimum requirements for the earthquake design and construction of buildings and building-like structures and/or their components subjected to earthquake ground motions.

1.2 Scope

1.2.1 Requirements of these provisions shall be applicable to reinforced concrete, steel and masonry buildings and building-like structures.

1.2.2 In addition to the buildings and building-like structures, non-building structures permitted to be designed in accordance with the requirements of these provisions are limited to those specified in Chapter 5.

In this context bridges, dams, harbor structures, tunnels, pipelines, power transmission lines, power generation plants including hydro, thermal and nuclear power plants, gas storage facilities, special defense installations, underground structures and other structures designed with analysis and safety requirements that are different than those for buildings are outside the scope of this code.

1.2.3 Requirements of these provisions shall not be applied to the buildings equipped with special systems and equipment between foundation and soil for the purpose of isolation of building structural system from the earthquake motion, and to the buildings incorporating other active or passive control systems.

1.2.4 Provisions to be applied to structures which are outside the scope of these provisions, shall be specifically determined by the Departments/Autonomous Organizations supervising the construction of such structures.

CHAPTER 2

SEISMIC HAZARD

2.1 Scope

This Chapter defines the minimum seismic hazard that has to be considered for the design of buildings.

2.2 Design Basis Ground Motion

Unless otherwise required, buildings shall be designed for a level of earthquake ground motion that has a 10% probability of exceedance in 50 years.

2.3 Seismic Zones

For the purpose of seismic design of buildings, Pakistan has been divided into five zones. These zones are based on the peak ground acceleration ranges summarized in Table 2.1.

The seismic zoning map of Pakistan is given in Fig. 2.1. The province/area-wise enlargements of the seismic zoning map of Pakistan are shown in Fig. 2.2 to 2.5.

Table 2.2 lists the seismic zones for all tehsils of the country.

The background for development of seismic zoning map of Pakistan is given in Appendix A.

2.4 Site-specific Hazard Analysis

The requirements of the seismic zoning map shall be superseded if a site-specific hazard analysis, probabilistic, deterministic or both, is carried out for a building or structure.

2.5 Modeling of Ground Motion

The results of site-specific seismic hazard analysis may be represented by response spectra and acceleration-time histories. The pertinent details are included in Chapter 5.

Table 2.1 - Seismic Zones

Seismic Zone	Peak Horizontal Ground Acceleration
1	0.05 to 0.08g
2A	0.08 to 0.16g
2B	0.16 to 0.24g
3	0.24 to 0.32g
4	> 0.32g

Where “g” is the acceleration due to gravity.

The acceleration values are for rock site condition with shear wave velocity (v_s) of 760 m/sec (Soil profile type S_B). For seismic zone factors, refer Table 5.9.

Sheet 1 of 4**Table 2.2 - Seismic Zones of Tehsils of Pakistan**

Tehsil	Seismic Zone	Tehsil	Seismic Zone	Tehsil	Seismic Zone
Balochistan					
Awaran	2B	Kachhi/Gandawa	2B	Loralai/Bori	3
Jhal Jao	3	Jhal Magsi	2B	Duki	3
Mshki (S/T)	3	Mirpur (S/T)	2B	Mekhtar (S/T)	3
Barkhan	3	Kalat	3	Mastung	3
Bela	2B	Gazg (S/T)	3	Dasht	3
Dureji	2B	Johan (S/T)	3	Khad Koocha (S/T)	3
Gadani (S/T)	2B	Mangochar (S/T)	3	Kirdgap (S/T)	3
Hub	2B	Surab	2B	Musakhel	3
Kanraj	2B	Kech	2B	Kingri (S/T)	3
Lakhra	2B	Balnigor (S/T)	2B	Nasirabad/Chattar	3
Liari (S/T)	2B	Buleda (S/T)	2B	D.M. Jamali	2B
Sonmiani/Winder	2B	Dasht (S/T)	3	Tamboo	3
Uthal	2B	Hoshab (S/T)	2B	Panjgur	2B
Bolan/Dhadar	3	Mand (S/T)	2B	Gichk (S/T)	2B
Balanari (S/T)	3	Tump	2B	Gowargo	2A
Bhag	3	Zamuran (S/T)	2B	Parome (S/T)	2B
Khattan (S/T)	3	Kharan	3	Pishin	4
Mach	3	Besima (S/T)	2B	Barshore (S/T)	3
Sani (S/T)	3	Mashkhel (S/T)	2A	Bostan (S/T)	4
Chagai (S/T)	2A	Nag (S/T)	2B	Hurramzai (S/T)	4
Dalbandin	2A	Wasuk (S/T)	2B	Karezat (S/T)	4
Nokundi (S/T)	2A	Khuzdar	2B	Quetta	4
Nushki	4	Aranji (S/T)	2B	Panjpai (S/T)	3
Taftan	2A	Karakh (S/T)	2B	Sibi	3
Dera Bugti	3	Moola (S/T)	2B	Kutmandai (S/T)	3
Baiker (S/T)	3	Nal (S/T)	3	Lehri	3
Loti	3	Ornach (S/T)	3	Sangan (S/T)	3
Malam (S/T)	3	Saroona (S/T)	2B	Ziarat	4
Phelawagh	3	Wadh (S/T)	2B	Harnai	3
Pir Koh (S/T)	3	Zehri	2B	Sinjawi (S/T)	4
Sangsillah (S/T)	3	Killa Abdullah	3	Zhab	3
Sui	3	Chaman	3	Ashwat (S/T)	2B
Gwadar	3	Dobandi (S/T)	3	Qamar Din Karez	2B
Jiwani	2B	Gulistan (S/T)	3	Sambaza (S/T)	3
Ormara	3	Killa Saifullah	3	Sherani (S/T)	3
Pasni	3	Baddini (S/T)	3		
Suntsar (S/T)	2B	Loiband (S/T)	3		
Jaffarabad/Jhat Pat	2B	Muslim Bagh	4		
Gandaka (S/T)	2B	Kohlu	3		
Panhwar (S/T)	2B	Kahan	3		
Usta Mohammad	2B	Mawand	3		

Sheet 2 of 4**Table 2.2 - Seismic Zones of Tehsils of Pakistan**

Tehsil	Seismic Zone	Tehsil	Seismic Zone	Tehsil	Seismic Zone
NWFP / FATA					
Abbottabad	3	Hangu	2B	Mir Ali	2B
Bajaur		Haripur	2B	Miran Shah	2B
Barang	3	Ghazi	2B	Razmak	2B
Charmang	3	Karak	2B	Shewa	2B
Khar Bajaur	3	Banda Daud Shah	2B	Spinwam	2B
Mamund	3	Takht-e-Nasrati	2B	Nowshera	2B
Nawagai	3	Khyber		Orakzai	
Salarzai	3	Bara	2B	Central Orakzai	2B
Utmankhel (Qzafi)	3	Jamrud	2B	Ismailzai	2B
Bannu	2B	Landi Kotal	3	Lower Orakzai	2B
Batagram	3	Mula Ghori	3	Upper Orakzai	2B
Allai	3	Kohat	2B	Peshawar	2B
Buner/Daggar	2B	Lachi	2B	Shangla/Alpuri	3
Charsadda	2B	Kurram		Besham	3
Tangi	3	Kurram F.R.	2B	Chakesar	3
Chitral	4	Lower Kurram	2B	Martung	3
Drosh	3	Upper Kurram	2B	Puran	2B
Lutkoh	3	Lakki Marwat	2B	South Waziristan	
Mastuj	3	Malakand/Swat		Birmal	2B
		Ranizai	3		
Mulkoh	3	Sam Ranizai	2B	Ladha	2B
Turkoh	3	Mansehra	3	Makin (Charlai)	2B
Dassu	3	Balakot	4	Sararogha	2B
Palas	3	Oghi	2B	Sarwekai	2B
Pattan	3	T.A.Adj. Mansehra Distt	3	Tiarza	2B
Dera Ismail Khan	2A	Mardan	2B	Toi Khullah	2B
Daraban	3	Takht Bhai	2B	Wana	2B
Kulachi	2B	Mohmand		Swabi	2B
Paharpur	2B	Halimzai	3	Lahore	2B
Dir	3	Pindiali	3	Swat	
Adenzai	3	Pringhar	3	Matta	3
Balambat	3	Safi	3	Tank	2B
Barawal	3	Upper Mohmand	3		
Khall	3	Utman Khel (Ambar)	3		
Kohistan	3	Yake Ghund	3		
Lalqila	3	North Waziristan			
Munda	3	Datta Khel	2B		
Samarbagh (Barwa)	3	Dossali	2B		
Temergara	3	Garyum	2B		
Wari	3	Ghulam Khan	2B		

Sheet 3 of 4

Table 2.2 - Seismic Zones of Tehsils of Pakistan

Tehsil	Seismic Zone	Tehsil	Seismic Zone	Tehsil	Seismic Zone
Punjab					
Attock	2B	Chiniot	2A	Pakpattan	2A
Fateh Jang	2B	Shorkot	2A	Arifwala	2A
Hassan Abdal	2B	Jhelum	2B	Rahim Yar Khan	2A
Jand	2B	Dina	2B	Khanpur	2A
Pindi Gheb	2B	Pind Dadan Khan	2B	Liaquatpur	2A
Bahawalnagar	2A	Sohawa	2B	Sadiqabad	2A
Chishtian	2A	Kasur	2A	Rajanpur	2A
Fort Abbas	1	Chunian	2A	De-Ex. Area of Rajanpur	2B
Haroonabad	2A	Pattoki	2A	Jampur	2A
Minchinabad	2A	Khanewal	2A	Rojhan	2A
Bahawalpur	2A	Jehanian	2A	Rawalpindi	2B
Ahmadpur East	2A	Kabirwala	2A	Gujar Khan	2B
Hasilpur	2A	Mian Channu	2A	Kahuta	2B
Khairpur Tamewali	2A	Khushab	2B	Kotli Sattian	3
Yazman	2A	Nurpur	2A	Murree	3
Bhakkar	2A	Lahore City	2A	Taxila	2B
Darya Khan	2A	Lahore Cantt	2A	Sahiwal	2A
Kalur Kot	2B	Leiah	2A	Chichawatni	2A
Mankera	2A	Chaubara	2A	Sargodha	2A
Chakwal	2B	Karor Lal Esan	2A	Bhalwal	2A
Choa Saidan Shah	2B	Lodhran	2A	Kot Momin	2A
Talagang	2B	Dunyapur	2A	Sahiwal	2A
Dera Ghazi Khan	2A	Kahrer Pacca	2A	Shahpur	2B
De-Ex.Area of D.G.Khan	2B	Mandi Bahauddin	2B	Sillanwali	2A
Taunsa	2B	Malikwal	2B	Sheikhupura	2A
Faisalabad City	2A	Phalia	2A	Ferozwala	2A
Chak Jhumra	2A	Mianwali	2B	Muridke	2A
Faisalabad Saddar	2A	Isa Khel	2B	Nankana Sahib	2A
Jaranwala	2A	Piplan	2B	Safdarabad	2A
Sammundri	2A	Multan City	2A	Sangla Hill	2A
Tandlianwala	2A	Jalalpur Pirwala	2A	Shahkot	2A
Gujranwala City	2A	Multan Saddar	2A	Sialkot	2B
Gujranwala Saddar	2A	Shujabad	2A	Daska	2B
Kamoki	2B	Muzaffargarh	2A	Pasrur	2B
Nowshera Virkan	2A	Alipur	2A	Toba Tek Singh	2A
Wazirabad	2A	Jatoi	2A	Gojra	2A
Gujrat	2B	Kot Addu	2A	Kamalia	2A
Kharian	2B	Narowal	2B	Vehari	2A
Sarai Alamgir	2B	Shakargarh	2B	Burewala	2A
Hafizabad	2A	Okara	2A	Mailsi	2A
Pindi Bhattian	2A	Depalpur	2A		
Jhang	2A	Renala Khurd	2A		

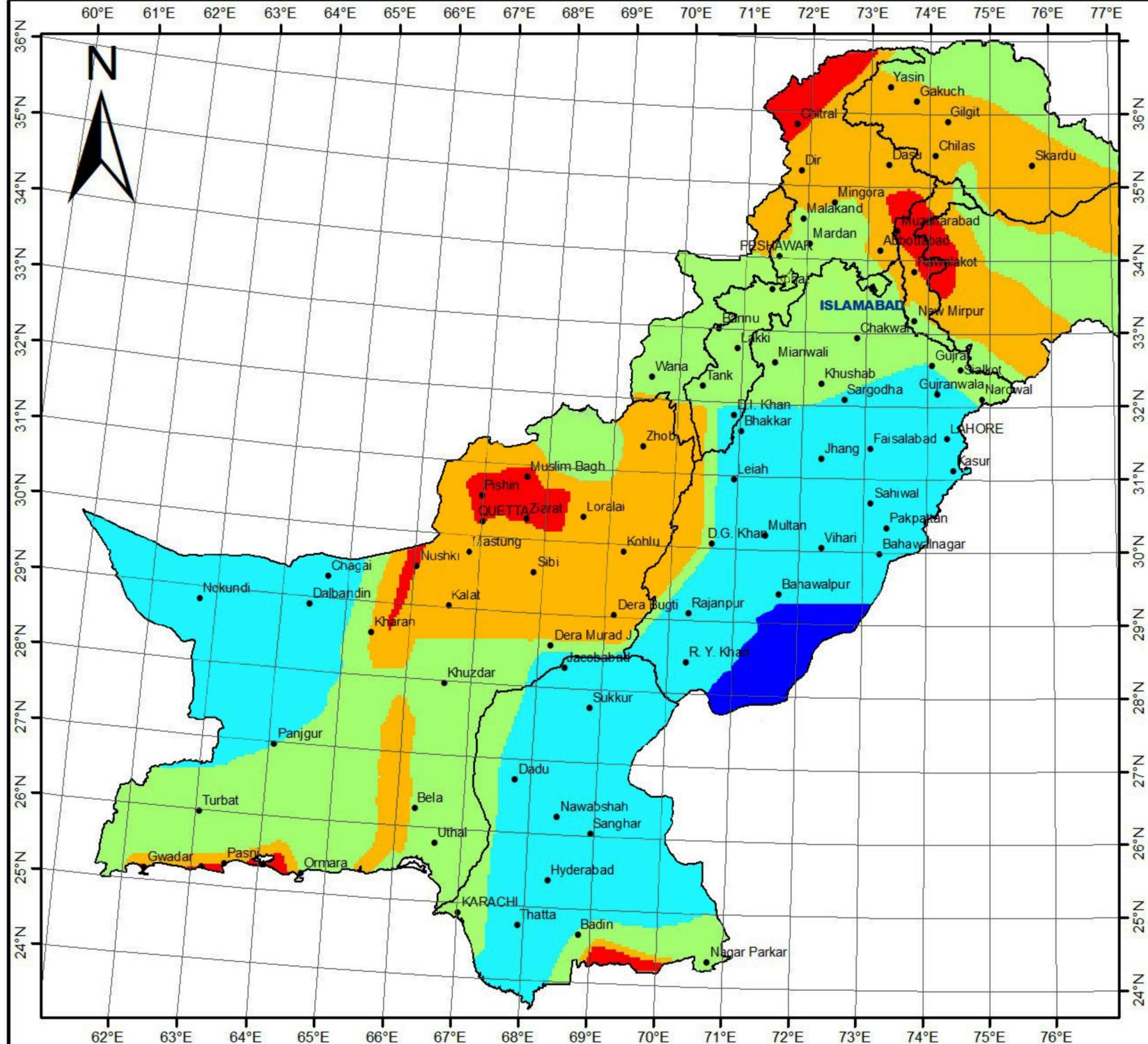
Sheet 4 of 4

Table 2.2 - Seismic Zones of Tehsils of Pakistan

Tehsil	Seismic Zone	Tehsil	Seismic Zone	Tehsil	Seismic Zone
Sindh					
Badin	2B	Dokri	2A	Shah Bunder	2A
Golarchi	2A	Kambar	2B	Sujawal	2A
Matli	2A	Miro Khan	2A	Umerkot	2A
Talhar	2B	Rato Dero	2A	Kunri	2A
Tando Bagho	2B	Shahdadkot	2B	Pithoro	2A
Dadu	2A	Warah	2A	Samaro	2A
Johi	2B	Malir	2B		
Khairpur Nathan Shah	2B	Mirpur Khas	2A	FEDERAL AREA	
Kotri	2A	Digri	2A	Islamabad	2B
Mehar	2A	Kot Ghulam Moh	2A		
Sehwan	2A	Naushahro Feroze	2A	AJ&K	
Ghotki	2A	Bhiria	2A	Bagh	4
Dharaki	2A	Kandioro	2A	Bhimbar	2B
Khangerh	2A	Moro	2A	Hajira	4
Mirpur Mathelo	2A	Nawab Shah	2A	Kotli	3
Ubauro	2A	Daulatpur	2A	Muzaffarabad	4
Hyderabad City	2A	Skrand	2A	New Mirpur	2B
Hala	2A	Sanghar	2A	Palandri	3
Hyderabad	2A	Jam Nawaz Ali	2A	Rawalakot	3
Latifabad	2A	Khipro	2A		
Matiari	2A	Shahdadpur	2A		
Qasimabad	2A	Sinjhoro	2A		
Tando Allahyar	2A	Tando Adam	2A	NORTHERN AREAS	
Tando Mohammad Khan	2A	Shikarpur	2A	Chilas	3
Jacobabad	2A	Garhi Yasin	2A	Dasu	3
Garhi Khairo	2A	Khanpur	2A	Gakuch	3
Kandhkot	2A	Lakhi	2A	Gilgit	3
Kashmor	2A	Sukkur	2A	Ishkuman	2B
Thul	2A	Pano Aqil	2A	Skardu	3
Karachi Central	2B	Rohri	2A	Yasin	3
Karachi East	2B	Salehpur	2A		
Karachi South	2B	Tharparkar/Chachro	2A		
Karachi West	2B	Diplo	3		
Khairpur	2A	Mithi	2B		
Faiz Ganj	2A	Nagar Parkar	2B		
Gambat	2A	Thatta	2A		
Kingri	2A	Ghorabari	2A		
Kot Diji	2A	Jati	2A		
Mirwah	2A	Keti Bunder	2A		
Nara	2A	Kharo Chan	2A		
Sobhodero	2A	Mirpur Bathoro	2A		
Larkana	2A	Mirpur Sakro	2A		

FIG. 2.1

SEISMIC ZONING MAP OF PAKISTAN



Seismic Zones

- Zone 1
 - Zone 2A
 - Zone 2B
 - Zone 3
 - Zone 4
- Cities

For enlargement of this map refer Figs. 2.2 to 2.5.

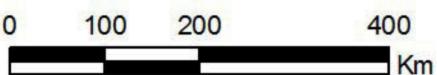


FIG. 2.2

SEISMIC ZONING MAP - BALOCHISTAN

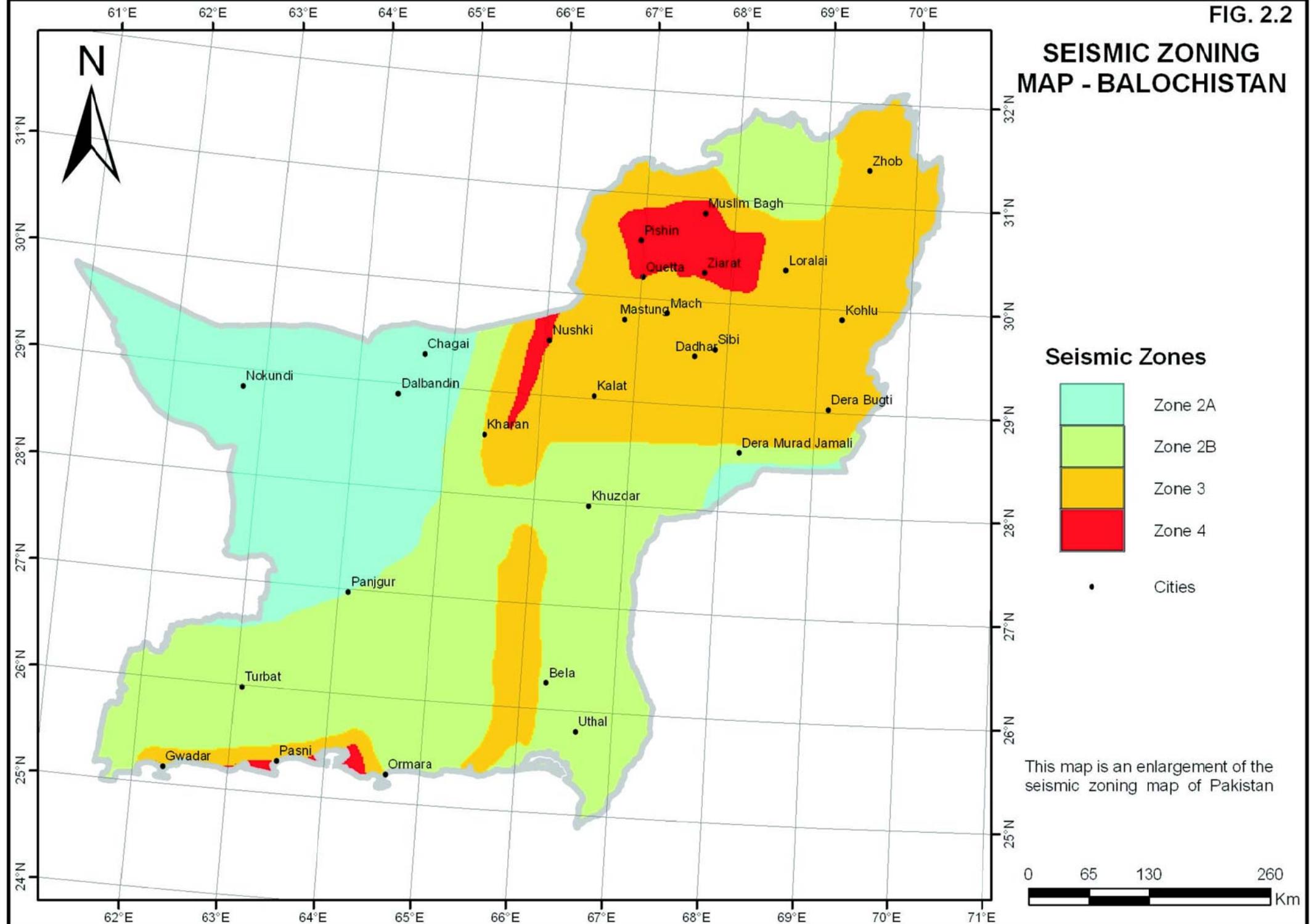


FIG. 2.3

SEISMIC ZONING MAP - SINDH

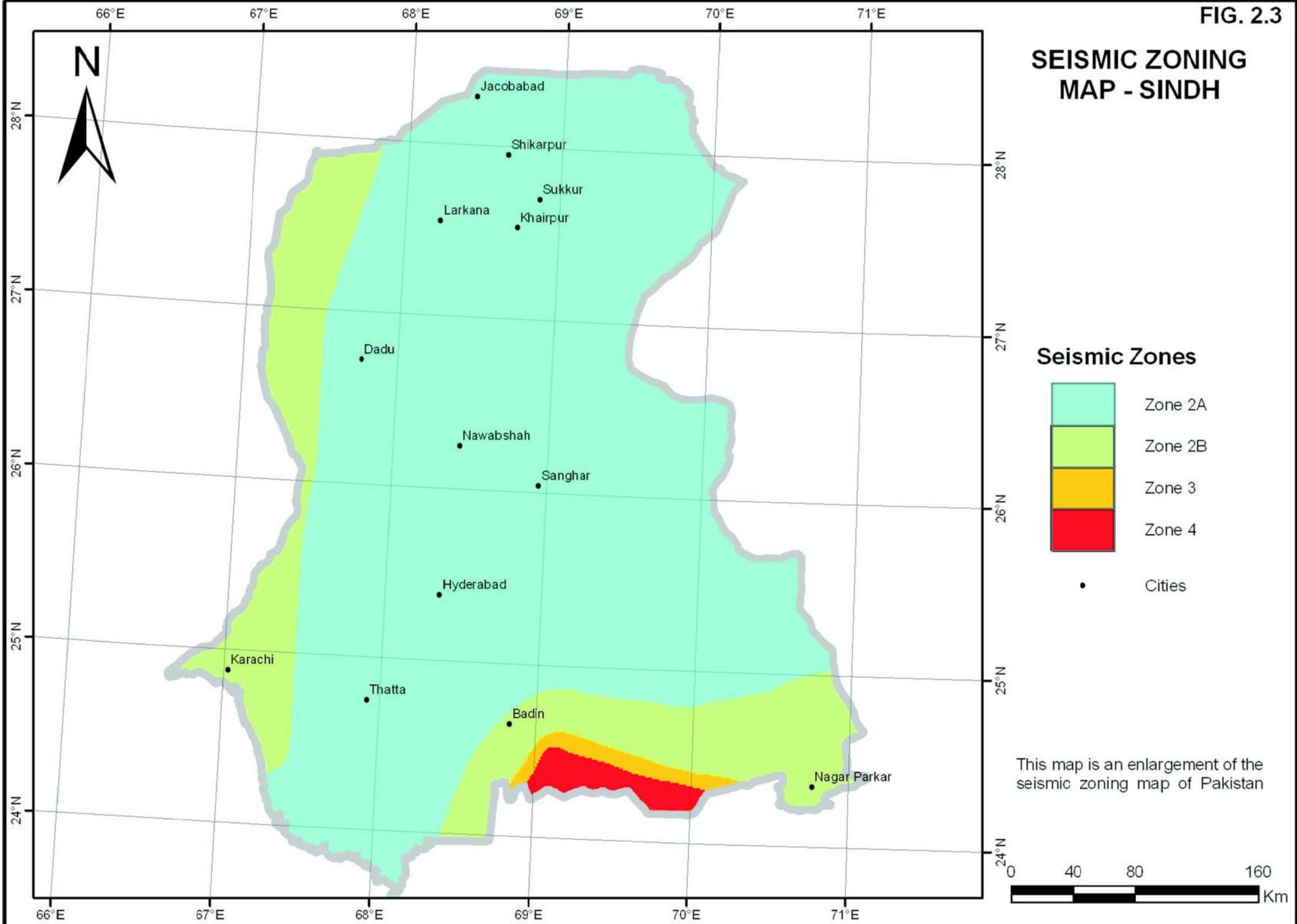
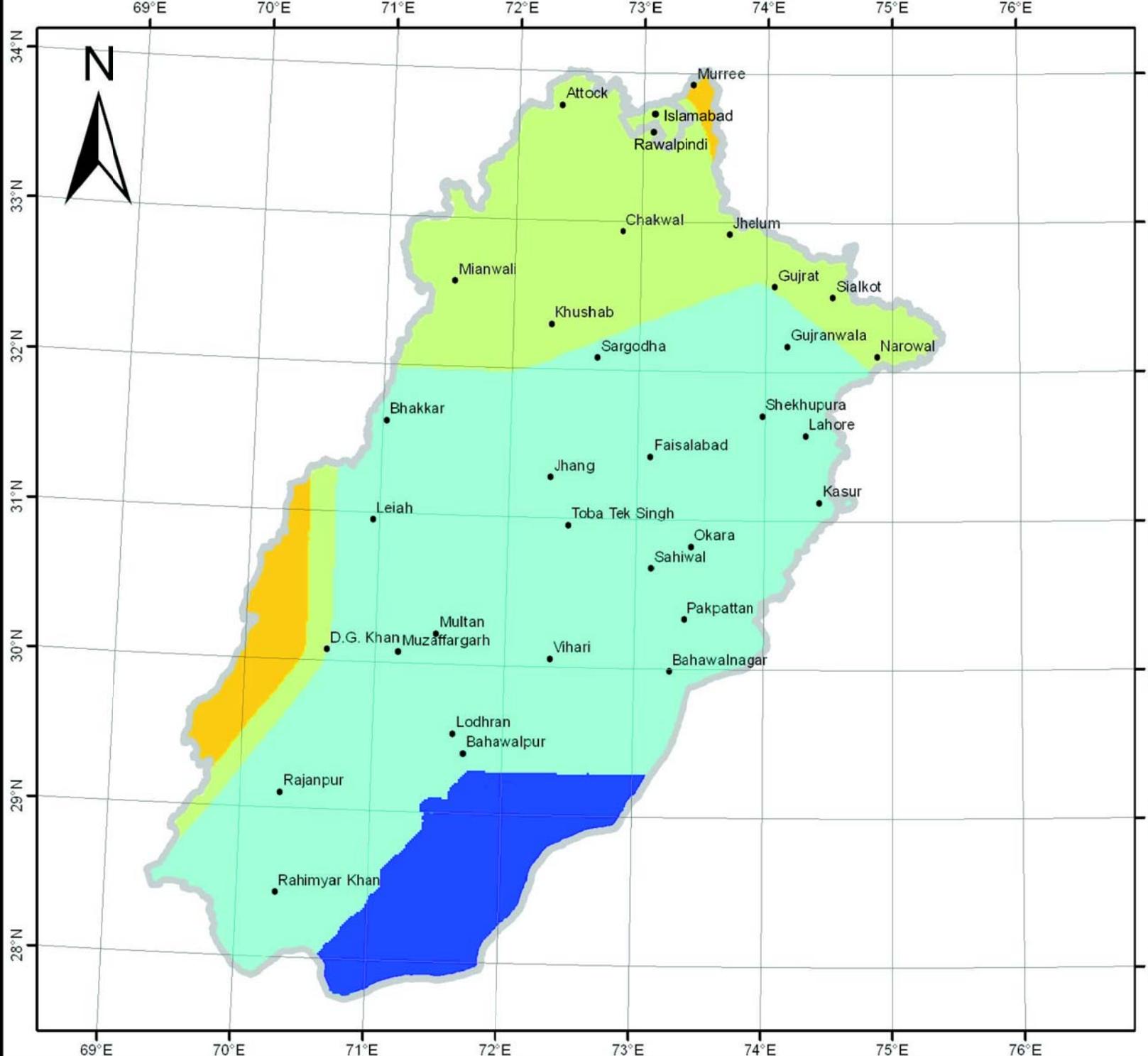


FIG. 2.4

SEISMIC ZONING MAP - PUNJAB



Seismic Zones

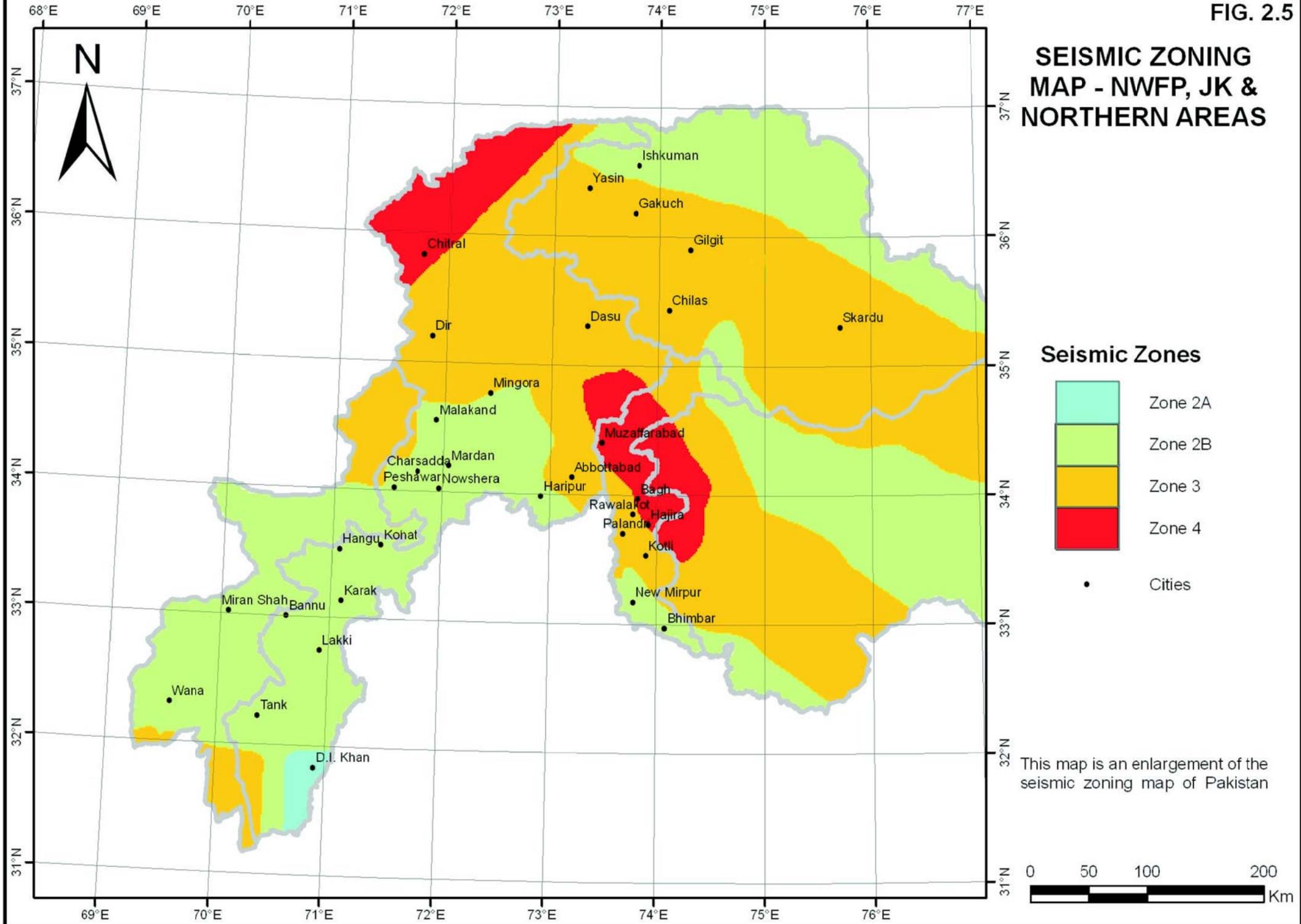
- Zone 1
- Zone 2A
- Zone 2B
- Zone 3
- Cities

This map is an enlargement of the seismic zoning map of Pakistan

0 45 90 180 Km

FIG. 2.5

SEISMIC ZONING MAP - NWFP, JK & NORTHERN AREAS



CHAPTER 3

SITE CONSIDERATIONS

3.1 Scope

The selection of suitable building sites shall be carried out based upon their geology/stratigraphy, distance from the causative fault, the liquefaction potential of site, earthquake induced landsliding, presence of sensitive clays and any other relevant geotechnical aspects, as provided in this chapter.

3.2 Potential Fault Rupture Hazard

An important building shall not be located within 200 meters (may vary with the earthquake magnitude) on either side of an active fault. However, areas closer than 200 meters to the trace of an active fault could be used for activities unlikely to be severely affected by surface faulting. These include use of such areas as grassland, forest, gardens, parks, small single storey specially designed dwellings etc.

3.3 Potential Liquefaction

The site selection for an important engineered building on potentially liquefiable soils shall be preceded by evaluation of liquefaction potential of the sub-surface through detailed geotechnical investigations and established analytical techniques. Necessary mitigation measures shall be taken to minimize the potential risk.

3.4 Potential Landslide and Slope Instability

Before deciding about placing a building on or adjacent to sloping ground in mountainous terrain, an examination of the hill slope stability conditions shall be made. The stability of sloping ground shall be evaluated and improvements, if required, shall be designed through an established analytical method.

On or adjacent to a sloping ground, the location of all buildings shall meet the requirements shown on Fig. 3.1, unless special slope stability measures are taken.

3.5 Sensitive Clays

The selection of site for a building on such soils shall be made on the basis of the detailed geotechnical investigations and adopting necessary mitigating measures in the structure and/or bearing ground.

CHAPTER 4

SOILS AND FOUNDATIONS

4.1 Symbols and Notations

d_c	=	Total thickness ($30 - d_s$) of cohesive soil layers in the top 30 m (100 ft).
d_i	=	Thickness of layer i , m (ft).
d_s	=	Total thickness of cohesionless soil layers in the top 30 m (100 ft).
F_t	=	That portion of the base shear, V , considered concentrated at the top of the structure in addition to F_n (Refer Chapter 5, Division IV).
H	=	Thickness of soil m (ft).
N_i	=	Standard penetration resistance of soil layer in accordance with ASTM D 1586.
N	=	Average field standard penetration resistance.
N_{CH}	=	Average standard penetration resistance for cohesion-less soil layers.
PI	=	Plasticity Index.
S_A, S_B	=	Soil profile types.
S_C, S_D		
S_E and S_F		
s_u	=	Undrained shear strength, kPa (psf).
s_{ui}	=	Undrained shear strength in accordance with approved nationally recognized standards, not to exceed 250 kPa (5,220 psf).
v_s	=	Measured shear wave velocity m/s (ft./sec).
v_{si}	=	Shear wave velocity in layer i , m/sec (ft/sec).
w_{mc}	=	Moisture content (%).

4.2 Scope

Determination of soil conditions of buildings to be constructed in seismic areas shall be performed, along with the applicable codes and standards, primarily in accordance with the rules and requirements of this section.

4.3 Determination of Soil Conditions

4.3.1 Site Geology and Soil Characteristics

4.3.1.1 Each site shall be assigned a soil profile type based on properly substantiated soil engineering characteristics using the site categorization procedure described in Section 4.4.

4.3.1.2 Soil Profile Types S_A, S_B, S_C, S_D and S_E are classified in Table 4.1. Soil Profile Type S_F is defined as soils requiring site specific evaluation.

4.4 Soil Profile Types

4.4.1 Scope

This section describes the procedure for determining Soil Profile Types S_A through S_F in accordance with Table 4.1.

4.4.2 Definitions

Soil profile types are defined as follows:

- S_A Hard rock with measured shear wave velocity, $v_s > 1500 \text{ m/s}$ ($4,920 \text{ ft/sec}$).
 S_B Medium hard rock with $750 \text{ m/s} < v_s < 1500 \text{ m/s}$ ($2,460 \text{ ft/sec} < v_s < 4,920 \text{ ft/sec}$).
 S_C Very dense soil and soft rock with $350 \text{ m/s} < v_s < 750 \text{ m/s}$ ($1,150 \text{ ft/sec} < v_s < 2,460 \text{ ft/sec}$) or with either $N > 50$ or $s_u > 100 \text{ kPa}$ ($2,088 \text{ psf}$).
 S_D Stiff soil with $175 \text{ m/s} < v_s < 350 \text{ m/s}$ ($575 \text{ ft/sec} < v_s < 1,150 \text{ ft/sec}$) or with $15 < N < 50$ or $50 \text{ kPa} < s_u < 100 \text{ kPa}$ ($1,044 \text{ psf} < s_u < 2,088 \text{ psf}$).
 S_E A soil profile with $v_s < 175 \text{ m/s}$ (575 ft/sec) or with $N < 15$ or with $s_u < 50 \text{ kPa}$ ($1,044 \text{ psf}$) or any profile with more than 3 m (10 ft) of soft clay defined as soil with $PI > 20$, $w_{mc} > 40$ percent and $s_u < 25 \text{ kPa}$ (522 psf).
 S_F Soils requiring site-specific evaluation:
1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils
 2. Peats and/or highly organic clays [$H > 3 \text{ m}$ (10 ft) of peat and/or highly organic clay where $H = \text{thickness of soil}$]
 3. Very high plasticity clays [$H > 7.5 \text{ m}$ (25 ft) with $PI > 75$]
 4. Very thick soft/medium stiff clays [$H > 37 \text{ m}$ (120 ft)]

Exception: When the soil properties are not known in sufficient detail to determine the soil profile type, generally Type S_D shall be used. Soil Profile Type S_E need not be assumed for all situations unless the engineer determines that Soil Profile Type S_E may be present at the site or in the event that Type S_E is established by geotechnical data.

The criteria set forth in the definition for Soil Profile Type S_F requiring site-specific evaluation shall be considered, where applicable. If the site corresponds to these criteria, the site shall be classified as Soil Profile Type S_F and a site-specific evaluation shall be conducted.

4.4.2.1 v_s method: Average shear wave velocity, v_s shall be determined in accordance with the following formula:

$$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (4.4-1)$$

4.4.2.2 N method: Average field standard penetration resistance (N) and average standard penetration resistance for cohesionless soil layers (N_{CH}). N and N_{CH} shall be determined in accordance with the following formulae:

$$N = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (4.4-2)$$

and

$$N_{CH} = \frac{d_s}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (4.4-3)$$

4.4.2.3 s_u method: Average undrained shear strength s_u , shall be determined in accordance with the following formula:

$$s_u = \frac{\sum_{i=1}^n d_c}{\sum_{i=1}^n \frac{d_i}{s_{ui}}} \quad (4.4-4)$$

4.4.2.4 Soft clay profile, S_E . The existence of a total thickness of soft clay greater than 3 m (10 ft) shall be investigated where a soft clay layer is defined by $s_u < 25$ kPa (522 psf), $w_{mc} > 40$ percent and $PI > 20$. If these criteria are met, the site shall be classified as Soil Profile Type S_E .

4.4.2.5 Soil profiles S_C , S_D and S_E . Sites with Soil Profile Types S_C , S_D and S_E shall be classified by using one of the following three methods with v_s , N and s_u computed in all cases as specified in Section 4.4.2.

1. v_s for the top 30 m (100 ft) (v_s method).
2. N for the top 30 m (100 ft) (N method).
3. N_{CH} for cohesionless soil layers ($PI < 20$) in the top 30 m (100 ft) and average s_u for cohesive soil layers ($PI > 20$) in top 30 m (100 ft) (s_u method).

4.4.2.6 Rock profiles, S_A and S_B . The shear wave velocity for medium rock, Soil Profile Type S_B , shall be either measured on site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type S_C .

The hard rock, Soil Profile Type S_A , category shall be supported by shear wave velocity measurement either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m (100 ft), surficial shear wave velocity measurements may be extrapolated to assess v_s . The rock categories, Soil Profile Types S_A and S_B , shall not be used if there is more than 3 m (10 ft) of soil between the rock surface and the bottom of the spread footing or mat foundation.

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

4.5 Foundation Construction in Seismic Zones 3 and 4

4.5.1 General

In Seismic Zones 3 and 4, requirements of this section in addition to other general requirements of Chapter 18, UBC 1997 shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto.

4.5.2 *Soil Capacity*

The foundation shall be capable of transmitting the design base shear and overturning forces prescribed in Section 5.30 from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

4.5.3 *Superstructure-to-Foundation Connection*

The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed.

4.5.4 *Foundation-Soil Interface*

For regular buildings, the force F_t as provided in Section 5.30.5 may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

4.5.5 *Special Requirements for Piles and Caissons***4.5.5.1** *General*

Piles, caissons and caps shall be designed according to the provisions of Section 5.30, including the effects of lateral displacements. Special detailing requirements as described in Section 4.5.5.2 shall apply for a length of piles equal to 120 percent of the flexural length. Flexural length shall be considered as a length of pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

4.5.5.2 *Non prestressed concrete piles and prestressed concrete piles*

4.5.5.2.1 *Non prestressed concrete piles.* Piles shall have transverse reinforcement meeting the requirements of Section 7.5.

Exception: Transverse reinforcement need not exceed the amount determined by Formula (7.5-2) in Section 7.5.4.1 for spiral or circular hoop reinforcement or by Formula (7.5-5) in Section 7.5.4.1 for rectangular hoop reinforcement.

4.5.5.2.2 *Prestressed concrete piles.* Piles shall have a minimum volumetric ratio of spiral reinforcement no less than 0.021 for 350 mm (14-inch) square and smaller piles, and 0.012 for 600 mm (24-inch) square and larger piles unless a smaller value can be justified by rational analysis. Interpolation may be used between the specified ratios for intermediate sizes.

Table 4.1-Soil Profile Types

Soil Profile Type	Soil Profile Name/ Generic Description	Average Properties for Top 30 M (100 ft) of Soil Profile		
		Shear Wave Velocity, v_s m/sec (ft/sec)	Standard Penetration Tests, N [or N_{CH} for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, s_u kPa (psf)
S_A	Hard Rock	>1,500 (>4,920)		
S_B	Rock	750 to 1,500 (2,460 to 4,920)	—	—
S_C	Very Dense Soil and Soft Rock	350 to 750 (1,150 to 2,460)	>50	>100 (>2,088)
S_D	Stiff Soil Profile	175 to 350 (575 to 1,150)	15 to 50	50 to 100 (1,044 to 2,088)
S_E^1	Soft Soil Profile	<175 (<575)	<15	<50 (<1,044)
S_F	Soil requiring Site-specific Evaluation. See 4.4.2			

1 Soil Profile Type S_E also includes any soil profile with more than 3 m (10 ft) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 25$ kPa (522 psf). The Plasticity Index, PI , and the moisture content, w_{mc} , shall be determined in accordance with the latest ASTM procedures.

CHAPTER 5

STRUCTURAL DESIGN REQUIREMENTS

Division I — General Design Requirements

5.1 Symbols and Notations

A	=	Area of floor or roof supported by the member, square meter (ft^2).
A_I	=	Influence area, in square meter (ft^2).
D	=	Dead load, kN/m^2 (psf).
E	=	Earthquake load set forth in Section 5.30.1, kN (lb).
E_m	=	Estimated maximum earthquake force that can be developed in the structure as set forth in Section 5.30.1.1, kN (lb).
F	=	Load due to fluids, kN (lb).
H	=	Load due to lateral pressure of soil and water in soil, kN (lb).
L	=	Live load, except roof live load, including any permitted live load reduction, kN/m^2 (psf).
L_o	=	Unreduced design live load per square meter (ft^2) of area supported by the member.
L_r	=	Roof live load, including any permitted live load reduction, kN/m^2 (psf).
P	=	Ponding load, kN/m^2 (psf).
R	=	Reduction in percentage.
r	=	Rate of reduction.
S	=	Snow load, kN/m^2 (psf).
T	=	Self-straining force and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof, kN (lb).
W	=	Load due to wind pressure, kN (lb).

5.2 Scope

This chapter prescribes general design requirements applicable to all structures regulated by this code.

5.3 Definitions

The following terms are defined for use in this code:

Allowable Stress Design is a method of proportioning structural elements such that computed stresses produced in the elements by the allowable stress load combinations do not exceed specified allowable stress (also called working stress design).

Balcony, Exterior is an exterior floor system projecting from a structure and supported by that structure, with no additional independent supports.

Dead Loads consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

Deck is an exterior floor system supported on at least two opposing sides by an adjoining structure and/or posts, piers, or other independent supports.

Factored Load is the product of a load specified in Sections 5.6 through 5.11 and a load factor. See Section 5.12.2 for combinations of factored loads.

Limit State is a condition in which a structure or component is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

Live Loads are those loads produced by the use and occupancy of the building or other structure and do not include dead load, construction load, or environmental loads such as wind load, snow load, rain load, earthquake load or flood load.

Load and Resistance Factor Design (LRFD) is a method of proportioning structural elements using load and resistance factors such that no applicable limit state is reached when the structure is subjected to all appropriate load combinations.

Strength Design is a method of proportioning structural elements such that the computed forces produced in the elements by the factored load combinations do not exceed the factored element strength.

5.4 Standards

The standards listed below are recognized standards and shall be referred where required:

Wind Design

1. ASCE 7, Chapter 6, Minimum Design Loads for Buildings and Other Structures.
2. ANSI EIA/TIA 222-E, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures.
3. ANSI/NAAMM FP1001, Guide Specifications for the Design Loads of Metal Flagpoles.

5.5 Design

5.5.1 General

Buildings and other structures and all portions thereof shall be designed and constructed to sustain, within the limitations specified in this code, all loads set forth in Chapter 5 and elsewhere in this code, combined in accordance with Section 5.12. Design shall be in accordance with Strength Design, Load and Resistance Factor Design or Allowable Stress Design methods, as permitted by the applicable chapters.

5.5.2 Rationality

Any system or method of construction to be used shall be based on a rational analysis in accordance with well established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load-resisting elements. The analysis shall include, but not be limited to, the provisions of Sections 5.5.2.1 through 5.5.2.3.

5.5.2.1 Distribution of horizontal shear. The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral-force-resisting system may be incorporated into buildings, provided that their effect on the action of the system is considered and provided for in the design.

Provision shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system. For accidental torsion requirements for seismic design, see Section 5.30.7.

5.5.2.2 *Stability against Overturning.* Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 5.11.6 for retaining walls, Section 5.16 for wind and Section 5.27 for seismic.

5.5.2.3 *Anchorage.* Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed forces.

Concrete and masonry walls shall be anchored to all floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than the minimum forces in Section 5.11.4. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 meter (4 feet). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 5.32, 5.33.2.8 and 5.33.2.9 for earthquake design requirements.

5.5.3 *Erection of Structural Framing*

Walls and structural framing shall be erected true and plumb in accordance with the design.

5.6 **Dead Loads**

5.6.1 *General*

Dead loads shall be as defined in Section 5.3 and in this section.

5.6.2 *Partition Loads*

Floors in office buildings and other buildings where partition locations are subject to change shall be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 1.0 kilo-Newton per square meter (21 psf) of floor area.

Exception: Access floor systems shall be designed to support, in addition to all other loads, a uniformly distributed dead load not less than 0.5 kilo-Newton per square meter (10.5 psf) of floor area.

5.7 **Live Loads**

5.7.1 *General*

Live loads shall be the maximum loads expected by the intended use or occupancy but in no case shall be less than the loads required by this section.

5.7.2 *Critical Distribution of Live Loads*

Where structural members are arranged to create continuity, members shall be designed using the loading conditions, which would cause maximum shear and bending moments. This requirement may be satisfied in accordance with the provisions of Section 5.7.3.2 or 5.7.4.2, where applicable.

5.7.3 *Floor Live Loads*

5.7.3.1 *General.* Floors shall be designed for the unit live loads as set forth in Table 5.1. These loads shall be taken as the minimum live loads in kilo-Newton per square meter (pounds per square foot) of horizontal projection to be used in the design of buildings for the occupancies listed, and loads at least equal shall be assumed for uses not listed in this section but that creates or accommodates similar loadings. Where it can be determined in designing floors that the actual live load will be greater than the value shown in Table 5.1, the actual live load shall be used in the design of such buildings or portions thereof. Special provisions shall be made for machine and apparatus loads.

5.7.3.2 *Distribution of uniform floor loads.* Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and alternate spans.

5.7.3.3 *Concentrated loads.* Provision shall be made in designing floors for a concentrated load, L , as set forth in Table 5.1 placed upon any space 0.75 meter (2.5 feet) square, wherever this load upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required thereof.

Provision shall be made in areas where vehicles are used or stored for concentrated loads, L , consisting of two or more loads spaced 1.50 meters (5 feet) nominally on center without uniform live loads. Each load shall be 40 percent of the gross weight of the maximum-size vehicle to be accommodated. Parking garages for the storage of private or pleasure-type motor vehicles with no repair or refueling shall have a floor system designed for a concentrated load of not less than 9 kilo-Newton (2,000 lbs) acting on an area of 0.013 square meter (20 in²) without uniform live loads. The condition of concentrated or uniform live load, combined in accordance with Section 5.12.2 or 5.12.3 as appropriate, producing the greatest stresses shall govern.

5.7.3.4 *Special loads.* Provision shall be made for the special vertical and lateral loads as set forth in Table 5.2.

5.7.3.5 *Live loads posted.* The live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed shall have such design live loads conspicuously posted by the owner in that part of each storey in which they apply, using durable metal signs, and it shall be unlawful to remove or deface such notices. The occupant of the building shall be responsible for keeping the actual load below the allowable limits.

5.7.4 *Roof Live Loads*

5.7.4.1 *General.* Roofs shall be designed for the unit live loads, L_r , set forth in Table 5.3. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

5.7.4.2 *Distribution of loads.* Where uniform roof loads are involved in the design of structural members arranged to create continuity, consideration may be limited to full dead loads on all spans in combination with full roof live loads on adjacent spans and on alternate spans.

Exception: Alternate span loading need not be considered where the uniform roof live load is 1.0 kilo-Newton per square meter (21 psf) or more or where load combinations, including snow load, result in larger members or connections.

For those conditions where light-gauge metal preformed structural sheets serve as the support and finish of roofs, roof structural members arranged to create continuity shall be considered adequate if designed for full dead loads on all spans in combination with the most critical one of the following superimposed loads:

1. Snow load in accordance with Section 5.14.
2. The uniform roof live load, L_r , set forth in Table 5.3 on all spans.
3. A concentrated gravity load, L_r , of 9.0 kilo-Newton (2,000 lbs) placed on any span supporting a tributary area greater than 18.5 square meter (200 ft²) to create maximum stresses in the member, whenever this loading creates greater stresses than those caused by the uniform live load. The concentrated load shall be placed on the member over a length of 0.75 meter (2.5 feet) along the span. The concentrated load need not be applied to more than one span simultaneously.
4. Water accumulation as prescribed in Section 5.11.7.

5.7.4.3 Unbalanced loading. Unbalanced loads shall be used where such loading will result in larger members or connections. Trusses and arches shall be designed to resist the stresses caused by unit live loads on one half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by the required unit live load on the entire span. For roofs whose structures are composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalanced unit live load design may be reduced by 50 percent.

5.7.4.4 Special roof loads. Roofs to be used for special purposes shall be designed for appropriate loads.

Greenhouse roof bars, purlins and rafters shall be designed to carry a 450 Newton (100 pounds) minimum concentrated load, L_r , in addition to the uniform live load.

5.7.5 Reduction of Live Loads

The design live load determined using the unit live loads as set forth in Table 5.1 for floors and Table 5.3, Method 2, for roofs may be reduced on any member supporting more than 14.0 square meter (150 ft²), including flat slabs, except for floors in places of public assembly and for live loads greater than 5.0 kilo-Newton per square meter (100 psf), in accordance with the following formula:

$$R = r(A - 14) \quad (5.7-1)$$

For FPS:

$$R = r(A - 150)$$

The reduction shall not exceed 40 percent for members receiving load from one level only, 60 percent for other members or R , as determined by the following formula:

$$R = 23.1(1 + D/L) \quad (5.7-2)$$

Where:

A = area of floor or roof supported by the member, square meter (ft²).

- D = dead load per square meter (ft^2) of area supported by the member.
 L = unit live load per square meter (ft^2) of area supported by the member.
 R = reduction in percentage.
 r = rate of reduction equal to 0.08 percent for floors. See Table 5.3 for roofs.

For storage loads exceeding 5.0 kilo-Newton per square meter (100 psf), no reduction shall be made, except that design live loads on columns may be reduced by 20 percent.

The live load reduction shall not exceed 40 percent in garages for the storage of private cars having a capacity of not more than nine passengers per vehicle.

5.7.6 *Alternate Floor Live Load Reduction*

As an alternate to Formula (5.7-1), the unit live loads set forth in Table 5.1 may be reduced in accordance with Formula (5.7-3) on any member, including flat slabs, having an influence area of 37.0 square meter (400 ft^2) or more.

$$L = L_o \left(0.25 + \frac{4.57}{\sqrt{A_I}} \right) \quad (5.7-3)$$

For FPS:

$$L = L_o \left(0.25 + \frac{15}{\sqrt{A_I}} \right)$$

Where:

- A_I = influence area, in square meter (ft^2). The influence area A_I is four times the tributary area for a column, two times the tributary area for a beam, equal to the panel area for a two-way slab, and equal to the product of the span and the full flange width for a precast T-beam.
 L = reduced design live load per square meter (ft^2) of area supported by the member.
 L_o = unreduced design live load per square meter (ft^2) of area supported by the member (Table 5.1).

The reduced live load shall not be less than 50 percent of the unit live load L_o for members receiving load from one level only, nor less than 40 percent of the unit live load L_o for other members.

5.8 Snow Loads

Snow loads shall be determined in accordance with Division II.

5.9 Wind Loads

Wind loads shall be determined in accordance with Division III.

5.10 Earthquake Loads

Earthquake loads shall be determined in accordance with Division IV.

5.11 Other Minimum Loads

5.11.1 General

In addition to the other design loads specified in this chapter, structures shall be designed to resist the loads specified in this section and the special loads set forth in Table 5.2.

5.11.2 Other Loads

Buildings and other structures and portions thereof shall be designed to resist all loads due to applicable fluid pressures, F , lateral soil pressures, H , ponding loads, P , and self-straining forces, T . See Section 5.11.7 for ponding loads for roofs.

5.11.3 Impact Loads

Impact loads shall be included in the design of any structure where impact loads occur.

5.11.4 Anchorage of Concrete and Masonry Walls

Concrete and masonry walls shall be anchored as required by Section 5.5.2.3. Such anchorage shall be capable of resisting the load combinations of Section 5.12.2 or 5.12.3 using the greater of the wind or earthquake loads required by this chapter or a minimum horizontal force of 4.1 kilo-Newton per linear meter (280.84 lb/ft) of wall, substituted for E .

5.11.5 Interior Wall Loads

Interior walls, permanent partitions and temporary partitions that exceed 1.85 meters (6.0 feet) in height shall be designed to resist all loads to which they are subjected but not less than a load, L , of 0.25 kilo-Newton per square meter (5.25 psf) applied perpendicular to the walls. The 0.25 kilo-Newton per square meter (5.25 psf) load need not be applied simultaneously with wind or seismic loads. The deflection of such walls under a load of 0.25 kilo-Newton per square meter (5.25 psf) shall not exceed 1/240 of the span for walls with brittle finishes and 1/120 of the span for walls with flexible finishes. See Table 5.14 for earthquake design requirements where such requirements are more restrictive.

Exception: Flexible, folding or portable partitions are not required to meet the load and deflection criteria but must be anchored to the supporting structure to meet the provisions of this code.

5.11.6 Retaining Walls

Retaining walls shall be designed to resist loads due to the lateral pressure of retained material in accordance with accepted engineering practice. Walls retaining drained soil, where the surface of the retained soil is level, shall be designed for a load, H , equivalent to that exerted by a fluid weighing not less than 4.75 kN/m²/m (30.25 psf per foot of depth) and having depth equal to that of the retained soil. Any surcharge shall be in addition to the equivalent fluid pressure.

Retaining walls shall be designed to resist sliding by at least 1.5 times the lateral force and overturning by at least 1.5 times the overturning moment, using allowable stress design loads.

5.11.7 Water Accumulation

All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead load or shall be designed to resist ponding load, P , combined in

accordance with Section 5.12.2 or 5.12.3. Ponding load shall include water accumulation from any source, including snow, due to deflection. See Section 1506 of UBC 1997 and Table 5.3, Footnote 3 of this chapter, for drainage slope. See Section 5.13 for deflection criteria.

5.11.8 *Hydrostatic Uplift*

All foundations, slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift load, F , equal to the full hydrostatic pressure.

5.11.9 *Flood-resistant Construction.* For flood-resistant construction requirements, where specifically adopted, the relevant standard codes may be consulted.

5.11.10 *Heliport and Helistop Landing Areas*

In addition to other design requirements of this chapter, heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with Section 5.12.2 or 5.12.3:

1. Dead load plus actual weight of the helicopter.
2. Dead load plus a single concentrated impact load, L , covering 0.1 square meters (1.0 ft^2) of 0.75 times the fully loaded weight of the helicopter if it is equipped with hydraulic-type shock absorbers, or 1.5 times the fully loaded weight of the helicopter if it is equipped with a rigid or skid-type landing gear.
3. The dead load plus a uniform live load, L , of 5.0 kilo-Newton per square meter (100 psf). The required live load may be reduced in accordance with Section 5.7.5 or 5.7.6.

5.11.11 *Prefabricated Construction*

5.11.11.1 *Connections.* Every device used to connect prefabricated assemblies shall be designed as required by this code and shall be capable of developing the strength of the members connected, except in the case of members forming part of a structural frame designed as specified in this chapter. Connections shall be capable of withstanding uplift forces as specified in this chapter.

5.11.11.2 *Pipes and conduit.* In structural design, due allowance shall be made for any material to be removed for the installation of pipes, conduits or other equipment.

5.11.11.3 *Tests and inspections.* See Section 6.4 for requirements for tests and inspections of prefabricated construction.

5.12 *Combinations of Loads*

5.12.1 *General*

Buildings and other structures and all portions thereof shall be designed to resist the load combinations specified in Section 5.12.2 or 5.12.3 and, where required by Chapter 5 Division IV, the special seismic load combinations of Section 5.12.4.

The most critical effect can occur when one or more of the contributing loads are not acting. All applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations.

5.12.2 Load Combinations Using Strength Design or Load and Resistance Factor Design

5.12.2.1 Basic load combinations. Where Load and Resistance Factor Design (Strength Design) is used, structures and all portions thereof shall resist the most critical effects from the following combinations of factored loads:

$$\begin{aligned} & 1.4 D & (5.12-1) \\ & 1.2 D + 1.6 L + 0.5 (L_r \text{ or } S) & (5.12-2) \\ & 1.2 D + 1.6 (L_r \text{ or } S) + (f_1 L \text{ or } 0.8 W) & (5.12-3) \\ & 1.2 D + 1.3 W + f_1 L + 0.5 (L_r \text{ or } S) & (5.12-4) \\ & 1.2 D + 1.0 E + (f_1 L + f_2 S) & (5.12-5) \\ & 0.9 D \pm (1.0 E \text{ or } 1.3 W) & (5.12-6) \end{aligned}$$

Where:

- f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 5.0 kilo-Newton per square meter (100 psf), and for garage live load.
= 0.5 for other live loads.
- f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure.
= 0.2 for other roof configurations.

Exceptions:

1. Factored load combinations of this section multiplied by 1.1 for concrete and masonry where load combinations include seismic forces.
2. Where other factored load combinations are specifically required by the provisions of this code.

5.12.2.2 Other loads. Where F , H , P or T is to be considered in design, each applicable load shall be added to the above combinations factored as follows: 1.3 F , 1.6 H , 1.2 P and 1.2 T .

5.12.3 Load Combinations Using Allowable Stress Design

5.12.3.1 Basic load combinations. Where allowable stress design (working stress design) is used, structures and all portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$$\begin{aligned} & D & (5.12-7) \\ & D + L + (L_r \text{ or } S) & (5.12-8) \\ & D + (W \text{ or } E / 1.4) & (5.12-9) \\ & 0.9 D \pm E / 1.4 & (5.12-10) \\ & D + 0.75 [L + (L_r \text{ or } S) + (W \text{ or } E / 1.4)] & (5.12-11) \end{aligned}$$

Additional combinations for steel structures only:

$$\begin{aligned} & 0.6 D + W & (5.12-12) \\ & 0.6 D + 0.7 E & (5.12-13) \end{aligned}$$

No increase in allowable stresses shall be used with these load combinations except as specifically permitted elsewhere in this code.

5.12.3.2 Other loads. Where F , H , P or T is to be considered in design, each applicable load shall be added to the combinations specified in Sections 5.12.3.1.

5.12.4 *Special Seismic Load Combinations*

For both Allowable Stress Design and Strength Design, the following special load combinations for seismic design shall be used as specifically required by Chapter 5, Division IV.

$$1.2 D + f_1 L + 1.0 E_m \quad (5.12-14)$$

$$0.9 D \pm 1.0 E_m \quad (5.12-15)$$

Where:

- f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 5.0 kilo-Newton per square meter (100 psf), and for garage live load.
= 0.5 for other live loads.

5.13 Deflection

The deflection of any structural member shall not exceed the values set forth in Table 5.4, based on the factors set forth in Table 5.5. The deflection criteria representing the most restrictive condition shall apply. Deflection criteria for materials not specified shall be developed in a manner consistent with the provisions of this section. See Section 5.11.7 for camber requirements.

Division II — Snow Loads**5.14 Snow loads**

Buildings and other structures and all portions thereof that are subject to snow loading shall be designed to resist the snow loads, in accordance with the load combinations set forth in Section 5.12.2 or 5.12.3.

Potential unbalanced accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration shall be considered. Snow loads in excess of 1.0 kilo-Newton per square meter (21 psf) may be reduced for each degree of pitch over 20 degrees by R_s as determined by the formula:

$$R_s = \frac{S}{40} - 0.024 \quad (5.14-1)$$

For **FPS**:

$$R_s = \frac{S}{40} - \frac{I}{2}$$

Where:

R_s = snow load reduction in kilo-Newton per square meter (pounds per square foot) per degree of pitch over 20 degrees.

S = total snow load in kilo-Newton per square meter (pounds per square foot).

Division III — Wind Design

5.15 Symbols and Notations

The following symbols and notations apply to the provisions of this division:

C_e	=	Combined height, exposure and gust factor coefficient as given in Table 5.7.
C_q	=	Pressure coefficient for the structure or portion of structure under consideration as given in Table 5.8.
I_w	=	Importance factor as set forth in Table 5.10.
P	=	Design wind pressure, kN/m ² (psf).
q_s	=	Wind stagnation pressure at the standard height of 10 meters (33 feet) as set forth in Table 5.6, kN/m ² (psf).

5.16 General

Every building or structure and every portion thereof shall be designed and constructed to resist the wind effects determined in accordance with the requirements of this division where applicable or any other internationally recognized building code. Wind shall be assumed to come from any horizontal direction. No reduction in wind pressure shall be taken for the shielding effect of adjacent structures.

Structures sensitive to dynamic effects, such as buildings with a height-to-width ratio greater than five, structures sensitive to wind-excited oscillations, such as vortex shedding or icing, and buildings over 125 meters (410 feet) in height, shall be, and any structure may be, designed in accordance with approved international standards.

The provisions of this section do not apply to building and foundation systems in those areas subject to scour and water pressure by wind and wave action. Buildings and foundations subject to such loads shall be designed in accordance with approved international standards.

5.17 Definitions

The following definitions apply only to this division:

Basic Wind Speed is the fastest-mile wind speed associated with an annual probability of 0.02 measured at a point 10 meters (33 feet) above the ground for an area having exposure category C.

Exposure B has terrain with buildings, forest or surface irregularities, covering at least 20 percent of the ground level area extending 1.60 km (1 mile) or more from the site.

Exposure C has terrain that is flat and generally open, extending 0.80 km (1/2 mile) or more from the site in any full quadrant.

Exposure D represents the most severe exposure in areas with basic wind speeds of 130 km/h (80 mph) or greater and has terrain that is flat and unobstructed facing large bodies of water over 1.60 km (1 mile) or more in width relative to any quadrant of the building site. Exposure D extends inland from the shoreline 0.40 km (1/4 mile) or 10 times the building height, whichever is greater.

Fastest-Mile Wind Speed is the wind speed obtained from wind velocity maps prepared by the concerned authorities or from local meteorological data and is the highest sustained average wind speed based on the time required for a mile-long sample of air to pass a fixed point.

Openings are apertures or holes in the exterior wall boundary of the structure. All windows or doors or other openings shall be considered as openings unless such openings and their frames are specifically detailed and designed to resist the loads on elements and components in accordance with the provisions of this section.

Partially Enclosed Structure or Storey is a structure or storey that has more than 15 percent of any windward projected area open and the area of opening on all other projected areas is less than half of that on the windward projection.

Unenclosed Structure or Storey is a structure that has 85 percent or more openings on all sides.

5.18 Basic Wind Speed

The minimum basic wind speed at any site shall not be less than as specified by the local regulatory authority. Until detailed wind data is available, all the structures inland shall be designed to resist a minimum wind velocity of not less than 120 km per hour (75 mph) at a height of 10 meters (33 ft) and all the structures along the coast shall be designed to resist a wind velocity of not less than 130 km per hour (80 mph) at a height of 10 meters (33 ft).

5.19 Exposure

An exposure shall be assigned at each site for which a building or structure is to be designed.

5.20 Design Wind Pressures

Design wind pressures for buildings and structures and elements therein shall be determined for any height in accordance with the following formula:

$$P = C_e C_q q_s I_w \quad (5.20-1)$$

5.21 Primary Frames and Systems

5.21.1 General

The primary frames or load-resisting system of every structure shall be designed for the pressures calculated using Formula (5.20-1) and the pressure coefficients, C_q , of either Method 1 or Method 2. In addition, design of the overall structure and its primary load-resisting system shall conform to Section 5.5.

The base overturning moment for the entire structure, or for any one of its individual primary lateral-resisting elements, shall not exceed two thirds of the dead-load-resisting moment. For an entire structure with a height-to-width ratio of 0.5 or less in the wind direction and a maximum height of 18.30 meters (60 feet), the combination of the effects of uplift and overturning may be reduced by one third. The weight of earth superimposed over footings may be used to calculate the dead-load-resisting moment.

5.21.2 Method 1 (Normal Force Method)

Method 1 shall be used for the design of gabled rigid frames and may be used for any structure. In the Normal Force Method, the wind pressures shall be assumed to act simultaneously normal to all exterior surfaces. For pressures on roofs and leeward walls, C_e , shall be evaluated at the mean roof height.

5.21.3 *Method 2 (Projected Area Method)*

Method 2 may be used for any structure less than 60 meters (196.8 feet) in height except those using gabled rigid frames. This method may be used in stability determinations for any structure less than 60 meters (196.8 feet) high. In the Projected Area Method, horizontal pressures shall be assumed to act upon the full vertical projected area of the structure, and the vertical pressures shall be assumed to act simultaneously upon the full horizontal projected area.

5.22 Elements and Components of Structures

Design wind pressures for each element or component of a structure shall be determined from Formula (5.20-1) and C_q values from Table 5.8, and shall be applied perpendicular to the surface. For outward acting forces the value of C_e shall be obtained from Table 5.7 based on the mean roof height and applied for the entire height of the structure. Each element or component shall be designed for the more severe of the following loadings:

1. The pressures determined using C_q values for elements and components acting over the entire tributary area of the element.
2. The pressures determined using C_q values for local areas at discontinuities such as corners, ridges and eaves. These local pressures shall be applied over a distance from a discontinuity of 3.0 meters (10 feet) or 0.1 times the least width of the structure, whichever is less.

The wind pressures from Sections 5.21 and 5.22 need not be combined.

5.23 Open-Frame Towers

Radio towers and other towers of trussed construction shall be designed and constructed to withstand wind pressures specified in this section, multiplied by the shape factors set forth in Table 5.8.

5.24 Miscellaneous Structures

Greenhouses, lath houses, agricultural buildings or fences 3.60 meters (12 feet) or less in height shall be designed in accordance with Division III. However, three fourths of q_s , but not less than 0.50 kilo-Newton per square meter (10.44 psf), may be substituted for q_s in Formula (5.20-1). Pressures on local areas at discontinuities need not be considered.

5.25 Occupancy Categories

For the purpose of wind-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 5.10. Table 5.10 lists importance factors, I_w , for each category.

Division IV — Earthquake Design

5.26 Symbols and Notations

The following symbols and notations apply to the provisions of this division:

A_B	=	Ground floor area of structure in square meter (ft^2) to include area covered by all overhangs and projections.
A_c	=	The combined effective area, in square meter (ft^2), of the shear walls in the first storey of the structure.
A_e	=	The minimum cross-sectional area in any horizontal plane in the first storey, in square meter (ft^2) of a shear wall.
A_x	=	The torsional amplification factor at <i>Level x</i> .
a_p	=	Numerical coefficient specified in Section 5.32 and set forth in Table 5.14.
C_a	=	Seismic coefficient, as set forth in Table 5.16.
C_t	=	Numerical coefficient given in Section 5.30.2.2.
C_v	=	Seismic coefficient, as set forth in Table 5.17.
D	=	Dead load on a structural element.
D_e	=	The length, in meter (ft), of a shear wall in the first storey in the direction parallel to the applied forces.
E, E_h		
E_m, E_v	=	Earthquake loads set forth in Section 5.30.1.
F_i		
F_n, F_x	=	Design Seismic Force applied to <i>Level i</i> , <i>n</i> or <i>x</i> , respectively.
F_p	=	Design Seismic Forces on a part of the structure.
F_{px}	=	Design Seismic Force on a diaphragm.
F_t	=	That portion of the base shear, V , considered concentrated at the top of the structure in addition to F_n .
f_i	=	Lateral force at Level <i>i</i> for use in Formula (5.30-10).
g	=	Acceleration due to gravity.
h_p		
h_n, h_x	=	Height in meter (ft) above the base to <i>Level i</i> , <i>n</i> or <i>x</i> , respectively.
h_r	=	Structure roof elevation with respect to grade in meter (ft).
I	=	Importance factor given in Table 5.10.
I_p	=	Importance factor specified in Table 5.10.
L	=	Live load on a structural element.
<i>Level i</i>	=	Level of the structure referred to by the subscript <i>i</i> . “ <i>i</i> = 1” designates the first level above the base.
<i>Level n</i>	=	That level that is uppermost in the main portion of the structure.
<i>Level x</i>	=	That level that is under design consideration. “ <i>x</i> = 1” designates the first level above the base.
l_w	=	Length of wall in meter (ft).
M	=	Maximum moment magnitude.
N_a	=	Near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 5.18 and 5.20.
N_v	=	Near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 5.19 and 5.20.
PI	=	Plasticity index of soil determined in accordance with approved international standards.

R	=	Numerical coefficient representative of the inherent over strength and global ductility capacity of lateral force-resisting systems, as set forth in Table 5.13 or 5.15.
R_p	=	Component response modification factor.
r	=	A ratio used in determining ρ . See Section 5.30.1.
$S_A, S_B,$ $S_C, S_D,$ S_E, S_F	=	Soil profile types as set forth in Table 4.1.
T	=	Elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration.
V	=	The total design lateral force or shear at the base given by Formula (5.30-4), (5.30-5), (5.30-6), (5.30-7) or (5.30-11).
V_x	=	The design storey shear in Storey x .
W	=	The total seismic dead load defined in Section 5.30.1.1.
w_i, w_x	=	That portion of W located at or assigned to Level i or x , respectively.
W_p	=	The weight of an element or component.
w_{px}	=	The weight of the diaphragm and the element tributary thereto at Level x , including applicable portions of other loads defined in Section 5.30.1.1.
Z	=	Seismic zone factor as given in Table 5.9.
Δ_M	=	Maximum Inelastic Response Displacement, which is the total drift or total storey drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in Section 5.30.9.
Δ_s	=	Design Level Response Displacement, which is the total drift or total storey drift that occurs when the structure is subjected to the design seismic forces.
δ_i	=	Horizontal displacement at Level i relative to the base due to applied lateral forces, f , for use in Formula (5.30-10).
ρ	=	Redundancy/Reliability Factor given by Formula (5.30-3).
Ω_o	=	Seismic Force Amplification Factor, which is required to account for structural overstrength and set forth in Table 5.13.
δ_{avg}	=	Average of the displacements at the extreme points of the structure at Level x .
δ_{max}	=	Maximum displacement at Level x .
Φ	=	Resistance factor.

5.27 General

5.27.1 Purpose

The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function.

5.27.2 Minimum Seismic Design

Structures and portions thereof shall, as a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this division.

5.27.3 Seismic and Wind Design

When wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this section and referenced sections shall be followed.

5.28 Definitions

For the purposes of this division, certain terms are defined as follows:

Base is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

Base Shear, V , is the total design lateral force or shear at the base of a structure.

Bearing Wall System is a structural system without a complete vertical load-carrying space frame. See Section 5.29.6.2.

Boundary Element is an element at edges of openings or at perimeters of shear walls or diaphragms.

Braced Frame is an essentially vertical truss system of the concentric or eccentric type that is provided to resist lateral forces.

Building Frame System is an essentially complete space frame that provides support for gravity loads. See Section 5.29.6.3.

Cantilevered Column Element is a column element in a lateral-force-resisting system that cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces applied essentially at the top.

Collector is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral-force-resisting system.

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

Component, Equipment is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

Component, Flexible is a component, including its attachments, having a fundamental period greater than 0.06 second.

Component, Rigid is a component, including its attachments, having a fundamental period less than or equal to 0.06 second.

Concentrically Braced Frame is a braced frame in which the members are subjected primarily to axial forces.

Design Basis Ground Motion is that ground motion that has a 10 percent chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with dynamic properties representative of the site characteristics shall be used to represent this ground motion. The dynamic effects of the Design Basis Ground Motion may be represented by the Design Response Spectrum. See Section 5.31.2.

Design Response Spectrum is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures in accordance with Sections 5.30 and 5.31. This response spectrum may be either a site-specific spectrum based on geologic, tectonic, seismological and soil characteristics associated with a specific site or may be a spectrum constructed in accordance with the spectral

shape in Fig. 5.1 using the site-specific values of C_a and C_v and multiplied by the acceleration of gravity, 9.815 m/sec² (386.4 in/sec²). See Section 5.31.2.

Design Seismic Force is the minimum total strength design base shear, factored and distributed in accordance with Section 5.30.

Diaphragm is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical-resisting elements. The term “diaphragm” includes horizontal bracing systems.

Diaphragm or Shear Wall Chord is the boundary element of a diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

Diaphragm Strut (drag strut, tie, and collector) is the element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical-resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

Drift See “storey drift.”

Dual System is a combination of moment-resisting frames and shear walls or braced frames designed in accordance with the criteria of Section 5.29.6.5.

Eccentrically Braced Frame (EBF) is a steel braced frame designed in conformance with Section 8.15, Chapter 8.

Elastic Response Parameters are forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation, in accordance with Section 5.30.

Essential Facilities are those structures that are necessary for emergency operations subsequent to a natural disaster.

Flexible Element or system is one whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in Section 5.30.6.

Horizontal Bracing System is a horizontal truss system that serves the same function as a diaphragm.

Intermediate Moment-Resisting Frame (IMRF) is a concrete frame designed in accordance with Section 7.13, Chapter 7.

Lateral-Force-Resisting System is that part of the structural system designed to resist the Design Seismic Forces.

Moment – Resisting Frame is a frame in which members and joints are capable of resisting forces primarily by flexure.

Ordinary Braced Frame (OBF) is a steel-braced frame designed in accordance with the provisions of Section 8.14, Chapter 8.

Ordinary Moment – Resisting Frame (OMRF) is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

Orthogonal Effects are the earthquake load effects on structural elements common to the lateral-force-resisting systems along two orthogonal axes.

Overstrength is a characteristic of structures where the actual strength is larger than the design strength. The degree of overstrength is material and system dependent.

PA Effect is the secondary effect on shears, axial forces and moments of frame members induced by the vertical loads acting on the laterally displaced building system.

Shear Wall is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

Shear wall – Frame Interactive System uses combinations of shear walls and frames designed to resist lateral forces in proportion to their relative rigidities, considering interaction between shear walls and frames on all levels.

Soft Storey is one in which the lateral stiffness is less than 70 percent of the stiffness of the storey above. See Table 5.11.

Space Frame is a three-dimensional structural system, without bearing walls, composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor-bracing systems.

Special Concentrically Braced Frame (SCBF) is a steel-braced frame designed in conformance with the provisions of Section 8.13, Chapter 8.

Special Moment – Resisting Frame (SMRF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Chapters 7 and 8.

Special Truss Moment Frame (STMF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the provisions of Section 8.12, Chapter 8.

Storey is the space between levels. Storey x is the storey below *Level x*.

Storey Drift is the lateral displacement of one level relative to the level above or below.

Storey Drift Ratio is the storey drift divided by the storey height.

Storey Shear, V_x , is the summation of design lateral forces above the storey under consideration.

Strength is the capacity of an element or a member to resist factored load as specified in Chapter 5.

Structure is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

Subdiaphragm is a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

Vertical Load – Carrying Frame is a space frame designed to carry vertical gravity loads.

Wall Anchorage System is the system of elements anchoring the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including sub-diaphragms and continuous ties, as specified in Sections 5.33.2.8 and 5.33.2.9.

Weak Storey is one in which the storey strength is less than 80 percent of the storey above. See Table 5.11.

5.29 Criteria Selection

5.29.1 Basis for Design

The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, overstrength and ductility of the lateral-force-resisting system. The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 5.30, except as modified by Section 5.31.5.4. Where strength design is used, the load combinations of Section 5.12.2 shall apply. Where Allowable Stress Design is used, the load combinations of Section 5.12.3 shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 5.12.3 are utilized. One- and two-family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

5.29.2 Occupancy Categories

For purposes of earthquake resistant design, each structure shall be placed in one of the occupancy categories listed in Table 5.10. Table 5.10 assigns importance factors, I and I_p , and structural observation requirements for each category.

5.29.3 Site Geology and Soil Characteristics

Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Chapter 4 and Table 4.1.

Exception: When the soil properties are not known in sufficient detail to determine the soil profile type, generally Type S_D shall be used. Soil Profile Type S_E need not be assumed for all situations unless the engineer determines that Soil Profile Type S_E may be present at the site or in the event that Type S_E is established by geotechnical data.

5.29.3.1 Soil profile type. Soil Profile Types S_A , S_B , S_C , S_D and S_E are defined in Table 4.1 and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
2. Peats and/or highly organic clays [$H > 3$ m (10 ft.) of peat and/or highly organic clay where H = thickness of soil].
3. Very high plasticity clays [$H > 7.5$ m (25 ft.) with $PI > 75$].
4. Very thick soft/medium stiff clays [$H > 37$ m (120 ft.)].

5.29.4 Site Seismic Hazard Characteristics

Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

5.29.4.1 Seismic Zone. Each site shall be assigned a seismic zone in accordance with Chapter 2, Fig. 2.1. Each structure shall be assigned a seismic zone factor Z , in accordance with Table 5.9.

5.29.4.2 Seismic Zone 4 near-source factor. In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 5.18 and the Seismic Source Type set forth in Table 5.20. The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A , S_B , S_C or S_D .
2. $\rho = 1.0$.
3. Except in single-storey structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.
4. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 5.11, and Type 1 or 4 of Table 5.12.

5.29.4.3 Seismic response coefficients. Each structure shall be assigned a seismic coefficient, C_a , in accordance with Table 5.16 and a seismic coefficient, C_v , in accordance with Table 5.17.

5.29.5 Configuration Requirements

5.29.5.1 General. Each structure shall be designated as being structurally regular or irregular in accordance with Sections 5.29.5.2 and 5.29.5.3.

5.29.5.2 Regular structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 5.29.5.3.

5.29.5.3 Irregular structures

1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 5.11 and 5.12. All structures in Seismic Zone 1 and Occupancy Categories 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 5.11) and horizontal irregularities of Type 1 (Table 5.12).
2. Structures having any of the features listed in Table 5.11 shall be designated as if having a vertical irregularity.

Exception: Where no storey drift ratio under design lateral forces is greater than 1.3 times the storey drift ratio of the storey above, the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 5.11. The storey drift ratio for the top two storeys need not be considered. The storey drifts for this determination may be calculated neglecting torsional effects.

3. Structures having any of the features listed in Table 5.12 shall be designated as having a plan irregularity.

5.29.6 Structural Systems

5.29.6.1 General. Structural systems shall be classified as one of the types listed in Table 5.13 and defined in this section.

5.29.6.2 Bearing wall system. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

5.29.6.3 Building frame system. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

5.29.6.4 Moment-resisting frame system. A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

5.29.6.5 Dual system. A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.
2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.
3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

5.29.6.6 Cantilevered column system. A structural system relying on cantilevered column elements for lateral resistance.

5.29.6.7 Undefined structural system. A structural system not listed in Table 5.13.

5.29.6.8 Nonbuilding structural system. A structural system conforming to Section 5.34.

5.29.7 Height Limits

Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 5.13.

Exception: Regular structures may exceed these limits by not more than 50 percent for unoccupied structures, which are not accessible to the general public.

5.29.8 Selection of Lateral-force Procedure

5.29.8.1 General. Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 5.31.

5.29.8.2 Simplified static. The simplified static lateral-force procedure set forth in Section 5.30.2.3 may be used for the following structures of Occupancy Category 4 or 5:

1. Buildings of any occupancy (including single-family dwellings) not more than three storeys in height excluding basements that use light-frame construction.
2. Other buildings not more than two storeys in height excluding basements.

5.29.8.3 Static. The static lateral force procedure of Section 5.30 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Categories 4 and 5 in Seismic Zone 2.
2. Regular structures below 73 meters (240 feet) in height with lateral force resistance provided by systems listed in Table 5.13, except where Section 5.29.8.4, Item 4, applies.

3. Irregular structures not more than five storeys or 20 meters (65 feet) in height.
4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average storey stiffness of the lower portion is at least 10 times the average storey stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

5.29.8.4 Dynamic. The dynamic lateral-force procedure of Section 5.31 shall be used for all other structures, including the following:

1. Structures 73 meters (240 feet) or more in height, except as permitted by Section 5.29.8.3, Item 1.
2. Structures having a stiffness, weight or geometric vertical irregularity of Type 1, 2 or 3, as defined in Table 5.11, or structures having irregular features not described in Table 5.11 or 5.12, except as permitted by Section 5.30.4.2.
3. Structures over five storeys or 20 meters (65 feet) in height in Seismic Zones 3 and 4 not having the same structural system throughout their height except as permitted by Section 5.30.4.2.
4. Structures, regular or irregular, located on Soil Profile Type S_F that has a period greater than 0.7 second. The analysis shall include the effects of the soils at the site and shall conform to Section 5.31.2, Item 4.

5.29.9 System Limitations

5.29.9.1 Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 5.11, shall not be over two storeys or 9 meters (30 feet) in height where the weak storey has a calculated strength of less than 65 percent of the storey above.

Exception: Where the weak storey is capable of resisting a total lateral seismic force of Ω_o times the design force prescribed in Section 5.30.

5.29.9.2 Undefined structural systems. For undefined structural systems not listed in Table 5.13, the coefficient R shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R :

1. Dynamic response characteristics,
2. Lateral force resistance,
3. Overstrength and strain hardening or softening,
4. Strength and stiffness degradation,
5. Energy dissipation characteristics,
6. System ductility, and
7. Redundancy.

5.29.9.3 Irregular features. All structures having irregular features described in Table 5.11 or 5.12 shall be designed to meet the additional requirements of those sections referenced in the tables.

5.29.10 Alternative Procedures

5.29.10.1 General. Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

5.29.10.2 Seismic isolation. Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the building official and when special detailing

is used to provide results equivalent to those obtained by the use of conventional structural systems.

5.30 Minimum Design Lateral Forces and Related Effects

5.30.1 Earthquake Loads and Modeling Requirements

5.30.1.1 Earthquake loads. Structures shall be designed for ground motion producing structural response and seismic forces in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in Section 5.12.

$$E = \rho E_h + E_v \quad (5.30-1)$$

$$E_m = Q_o E_h \quad (5.30-2)$$

Where:

- E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, E_h , and the vertical component, E_v .
- E_h = the earthquake load due to the base shear, V , as set forth in Section 5.30.2 or the design lateral force, F_p , as set forth in Section 5.32.
- E_m = the estimated maximum earthquake force that can be developed in the structure as set forth in Section 5.30.1.1.
- E_v = the load effect resulting from the vertical component of the earthquake ground motion and is equal to an addition of $0.5 C_a I D$ to the dead load effect, D , for Strength Design, and may be taken as zero for Allowable Stress Design.
- Q_o = the seismic force amplification factor that is required to account for structural overstrength, as set forth in Section 5.30.3.1.
- ρ = Reliability/Redundancy Factor as given by the following formula:

$$\rho = 2 - \frac{6.1}{r_{\max} \sqrt{A_B}} \quad (5.30-3)$$

In FPS:

$$\rho = 2 - \frac{20}{r_{\max} \sqrt{A_B}}$$

Where:

- r_{\max} = the maximum element-storey shear ratio. For a given direction of loading, the element-storey shear ratio is the ratio of the design storey shear in the most heavily loaded single element divided by the total design storey shear. For any given Storey Level i , the element-storey shear ratio is denoted as r_i . The maximum element-storey shear ratio r_{\max} is defined as the largest of the element storey shear ratios, r_i , which occurs in any of the storey levels at or below the two-thirds height level of the building.

For braced frames, the value of r_i is equal to the maximum horizontal force component in a single brace element divided by the total storey shear.

For moment frames, r_i shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the storey shear. For columns common to

two bays with moment-resisting-connections on opposite sides at *Level i* in the direction under consideration, 70 percent of the shear in that column may be used in the column shear summation.

For shear walls, r_i shall be taken as the maximum value of the product of the wall shear multiplied by $3.05/l_w$ (For FPS: $10/l_w$) and divided by the total storey shear, where l_w is the length of the wall in meter (ft).

For dual systems, r_i shall be taken as the maximum value of r_i as defined above considering all lateral-load-resisting elements. The lateral loads shall be distributed to elements based on relative rigidities considering the interaction of the dual system. For dual systems, the value of ρ need not exceed 80 percent of the value calculated above.

ρ shall not be taken less than 1.0 and need not be greater than 1.5, and A_B is the ground floor area of the structure in square meter (ft^2). For special moment-resisting frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of special moment-resisting frames shall be increased to reduce r , such that ρ is less than or equal to 1.25.

Exception: A_B may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at the ground floor.

When calculating drift, or when the structure is located in Seismic Zone 1 or 2, ρ shall be taken equal to 1.

The ground motion producing lateral response and design seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by Section 5.33.1.

Seismic dead load, W , is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.
2. Where a partition load is used in the floor design, a load of not less than 0.50 kN/m^2 (10 psf) shall be included.
3. Design snow loads of 1.50 kN/m^2 (30 psf) or less need not be included. Where design snow loads exceed 1.50 kN/m^2 (30 psf), the design snow load shall be included, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the building official.
4. Total weight of permanent equipment shall be included.

5.30.1.2 Modeling requirements. The mathematical model of the physical structure shall include all elements of the lateral-force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, the model shall comply with the following:

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections.
2. For steel moment frame systems, the contribution of panel zone deformations to overall storey drift shall be included.

5.30.1.3 $P\Delta$ effects. The resulting member forces and moments and the storey drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall

be evaluated using the forces producing the displacements of Δ_s . $P\Delta$ effects need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any storey as the product of the total dead, floor live and snow load, as required in Section 5.12, above the storey times the seismic drift in that storey divided by the product of the seismic shear in that storey times the height of that storey. In Seismic Zones 3 and 4, $P\Delta$ effects need not be considered when the storey drift ratio does not exceed $0.02/R$.

5.30.2 Static Force Procedure

5.30.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{R T} W \quad (5.30-4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \quad (5.30-5)$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \quad (5.30-6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \quad (5.30-7)$$

5.30.2.2 Structure period. The value of T shall be determined from one of the following methods:

1. *Method A:* For all buildings, the value T may be approximated from the following formula:

$$T = C_t (h_n)^{\frac{3}{4}} \quad (5.30-8)$$

Where:

$C_t = 0.0853$ (0.035) for steel moment-resisting frames.

$C_t = 0.0731$ (0.030) for reinforced concrete moment-resisting frames and eccentrically braced frames.

$C_t = 0.0488$ (0.020) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $\frac{0.0743}{\sqrt{A_c}}$ (For FPS: $\frac{0.1}{\sqrt{A_c}}$ for A_c in ft^2). The value of A_c shall be determined from the following formula:

$$A_c = \sum A_e \left[0.2 + \left(\frac{D_e}{h_n} \right)^2 \right] \quad (5.30-9)$$

The value of D_e / h_n used in Formula (5.30-9) shall not exceed 0.9.

2. *Method B:* The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 5.30.1.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4 and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi \sqrt{\left(\sum_{i=1}^n w_i \delta_i^2 \right) \div \left(g \sum_{i=1}^n f_i \delta_i \right)} \quad (5.30-10)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (5.30-13), (5.30-14) and (5.30-15) or any other rational distribution. The elastic deflections, δ_i , shall be calculated using the applied lateral forces, f_i .

5.30.2.3 Simplified design base shear

5.30.2.3.1 General. Structures conforming to the requirements of Section 5.29.8.2 may be designed using this procedure.

5.30.2.3.2 Base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{3.0C_a}{R} W \quad (5.30-11)$$

Where the value of C_a shall be based on Table 5.16 for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zones 3 and 4, and Type S_E shall be used in Seismic Zones 1, 2A and 2B. In Seismic Zone 4, the Near-Source Factor, N_a , need not be greater than 1.3 if none of the following structural irregularities are present: Type 1, 4 or 5 of Table 5.11, or Type 1 or 4 of Table 5.12.

5.30.2.3.3 Vertical distribution. The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0C_a}{R} w_i \quad (5.30-12)$$

Where the value of C_a shall be determined in Section 5.30.2.3.2.

5.30.2.3.4 Applicability. Sections 5.30.1.2, 5.30.1.3, 5.30.2.1, 5.30.2.2, 5.30.5, 5.30.9, 5.30.10 and 5.31 shall not apply when using the simplified procedure.

Exception: For buildings with relatively flexible structural systems, the building official may require consideration of $P\Delta$ effects and drift in accordance with Sections 5.30.1.3, 5.30.9 and 5.30.10. A_s shall be prepared using design seismic forces from Section 5.30.2.3.2.

Where used, Δ_M shall be taken equal to 0.01 times the storey height of all storeys. In Section 5.33.2.9, Formula (5.33-1) shall read $F_{px} = \frac{3.0C_a}{R} w_{px}$ and need not exceed 1.0 $C_a w_{px}$, but shall not be less than 0.5 $C_a w_{px}$. R and Ω_o shall be taken from Table 5.13.

5.30.3 *Determination of Seismic Factors*

5.30.3.1 Determination of Ω_o . For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force overstrength factor Ω_o and the design seismic forces set forth in Section 5.30. For both Allowable Stress Design and Strength Design, the Seismic Force Overstrength Factor, Ω_o , shall be taken from Table 5.13.

5.30.3.2 Determination of R . The notation R shall be taken from Table 5.13.

5.30.4 *Combinations of Structural Systems*

5.30.4.1 General. Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

5.30.4.2 Vertical combinations. The value of R used in the design of any storey shall be less than or equal to the value of R used in the given direction for the storey above.

Exception: This requirement need not be applied to a storey where the dead weight above that storey is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

1. The entire structure is designed using the lowest R of the lateral-force-resisting systems used, or
2. The following two-stage static analysis procedures may be used for structures conforming to Section 5.29.8.3, Item 4.
 - 2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ .
 - 2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ . The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the (R / ρ) of the upper portion over (R / ρ) of the lower portion.

5.30.4.3 Combinations along different axes. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of R used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 49 meters (160 feet) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 49 meters (160 feet) in height in Seismic Zones 3 and 4.

5.30.4.4 Combinations along the same axis. For other than dual systems and shear wall-frame interactive systems in Seismic Zone 1, where a combination of different structural systems is

utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

5.30.5 Vertical Distribution of Force

The total force shall be distributed over the height of the structure in conformance with Formulas (5.30-13), (5.30-14) and (5.30-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^n F_i \quad (5.30-13)$$

The concentrated force F_t at the top, which is in addition to F_n , shall be determined from the formula:

$$F_t = 0.07TV \quad (5.30-14)$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (5.30-4). F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including *Level n*, according to the following formula:

$$F_x = (V - F_t) \frac{w_x h_x}{\sum_{i=1}^n (w_i h_i)} \quad (5.30-15)$$

At each level designated as x , the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

5.30.6 Horizontal Distribution of Shear

The design storey shear, V_x , in any storey is the sum of the forces F_t and F_x above that storey. V_x shall be distributed to the various elements of the vertical lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 5.33.2.4 for rigid elements that are not intended to be part of the lateral-force resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the storey shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of storey shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the storey drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

5.30.7 Horizontal Torsional Moments

Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given storey shall be the moment resulting from eccentricities between applied design lateral forces at levels above that storey and the vertical-resisting elements in that storey plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 5.30.6.

Where torsional irregularity exists, as defined in Table 5.12, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x , determined from the following formula:

$$A_x = \left[\frac{\delta_{\max}}{1.2\delta_{\text{avg}}} \right]^2 \quad (5.30-16)$$

Where:

δ_{avg} = the average of the displacements at the extreme points of the structure at *Level x*.
 δ_{\max} = the maximum displacement at *Level x*.

The value of A_x need not exceed 3.0.

5.30.8 Overturning

5.30.8.1 General. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 5.30.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 5.30.6. Overturning effects on every element shall be carried down to the foundation. See Sections 5.12 and 5.33 for combining gravity and seismic forces.

5.30.8.2 Elements supporting discontinuous systems

5.30.8.2.1 General. Where any portion of the lateral-load-resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 5.11 or plan irregularity Type 4 in Table 5.12, concrete, masonry, steel and wood elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 5.12.4.

Exceptions:

1. The quantity E_m in Section 5.12.4 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.
2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ , of 1.0.

5.30.8.2.2 *Detailing requirements in Seismic Zones 3 and 4.* In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete elements designed primarily as axial load members shall comply with Section 7.5.4.5, Chapter 7.
2. Steel elements designed primarily as axial-load members shall comply with Section 8.8.3, Chapter 8.
3. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system.

5.30.8.3 *At foundation.* See Sections 5.29.1 and 4.5.4 for overturning moments to be resisted at the foundation soil interface.

5.30.9 *Drift*

Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M , of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 5.30.2.1, Δ_S , shall be determined in accordance with Section 5.30.9.1. To determine Δ_M , these drifts shall be amplified in accordance with Section 5.30.9.2.

5.30.9.1 *Determination of Δ_S .* A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 5.30.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 5.31. Where Allowable Stress Design is used and where drift is being computed, the load combinations of Section 5.12.2 shall be used. The mathematical model shall comply with Section 5.30.1.2. The resulting deformations, denoted as Δ_S , shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

5.30.9.2 *Determination of Δ_M .* The Maximum Inelastic Response Displacement, Δ_M , shall be computed as follows:

$$\Delta_M = 0.7 R \Delta_S \quad (5.30-17)$$

Exception: Alternatively, Δ_M may be computed by nonlinear time history analysis in accordance with Section 5.31.6.

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider $P\Delta$ effects.

5.30.10 *Storey Drift Limitation*

5.30.10.1 *General.* Storey drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M .

5.30.10.2 *Calculated.* Calculated storey drift using Δ_M shall not exceed 0.025 times the storey height for structures having a fundamental period of less than 0.7 second. For structures having a

fundamental period of 0.7 second or greater, the calculated storey drift shall not exceed 0.020 times the storey height.

Exceptions:

1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement, Δ_M .
2. There shall be no drift limit in single-storey steel-framed structures classified as Groups B, F and S Occupancies or Group H, Division 4 or 5 Occupancies. In Groups B, F and S Occupancies, the primary use shall be limited to storage, factories or workshops. Structures on which this exception is used shall not have equipment attached to the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift in accordance with Section 5.33.2.4.

5.30.10.3 Limitations. The design lateral forces used to determine the calculated drift may disregard the limitations of Formula (5.30-6) and may be based on the period determined from Formula (5.30-10) neglecting the 30 or 40 percent limitations of Section 5.30.2.2, Item 2.

5.30.11 Vertical Component

The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7C_a I W_p$.

In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

5.31 Dynamic Analysis Procedures

5.31.1 General

Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

5.31.2 Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Fig. 5.1, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 9.815 m/sec^2 (386.4 in/sec^2).
2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be

consistent with the anticipated structural behavior at the intensity of shaking established for the site.

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 5.31.2, Item 2.
4. For structures on Soil Profile Type S_F , the following requirements shall apply when required by Section 5.29.8.4, Item 4:
 - 4.1 The ground motion representation shall be developed in accordance with Items 2 and 3.
 - 4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.
5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site specific data. Where the Near Source Factor, N_a , is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

5.31.3 Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 5.12 and having a rigid or semi-rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 5.30.1.2.

5.31.4 Description of Analysis Procedures

5.31.4.1 Response spectrum analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

5.31.4.2 Time-history analysis. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

5.31.5 Response Spectrum Analysis

5.31.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 5.31.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 5.31.5.4.

5.31.5.2 Number of modes. The requirement of Section 5.31.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

5.31.5.3 Combining modes. The peak member forces, displacements, storey forces, storey shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

5.31.5.4 Reduction of elastic response parameters for design

Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R .

1. For all regular structures where the ground motion representation complies with Section 5.31.2, Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 5.30.2.
2. For all regular structures where the ground motion representation complies with Section 5.31.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 5.30.2.
3. For all irregular structures, regardless of the ground motion representation, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 5.30.2.

The corresponding reduced design seismic forces shall be used for design in accordance with Section 5.12.

5.31.5.5 Directional effects. Directional effects for horizontal ground motion shall conform to the requirements of Section 5.30.1. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with Section 5.30.11. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

5.31.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 5.30.7. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 5.30.6.

5.31.5.7 Dual systems. Where the lateral forces are resisted by a dual system as defined in Section 5.29.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 5.29.6.5, Item 2, and may be analyzed using either the procedures of Section 5.30.5 or those of Section 5.31.5.

5.31.6 Time-history Analysis

5.31.6.1 Time-history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of

the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from $0.2T$ second to $1.5T$ seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

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

5.31.6.2 Elastic time-histories analysis. Elastic time history shall conform to Sections 5.31.1, 5.31.2, 5.31.3, 5.31.5.2, 5.31.5.4, 5.31.5.5, 5.31.5.6, 5.31.5.7 and 5.31.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 5.31.5.4.

5.31.6.3 Nonlinear time-history analysis

5.31.6.3.1 Nonlinear time-history. Nonlinear time-history analysis shall meet the requirements of Section 5.29.10, and time histories shall be developed and results determined in accordance with the requirements of Section 5.31.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 5.30.10.

5.31.6.3.2 Design review. When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral force resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.
2. Reviewing the preliminary design of the lateral-force-resisting system.
3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer of record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

5.32 Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures

5.32.1 General

Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 5.32.2. Attachments for floor- or roof-mounted equipment weighing less than 181 kilograms (400 pounds) and furniture need not be designed.

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of non-rigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 5.32.2.

When permissible design strengths and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from relevant international standards subject to the approval of the building official.

5.32.2 *Design for Total Lateral Force*

The total design lateral seismic force, F_p , shall be determined from the following formula:

$$F_p = 4.0C_a I_p W_p \quad (5.32-1)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p} \left(1 + \frac{3h_x}{h_r} \right) W_p \quad (5.32-2)$$

Except that:

$$F_p \text{ shall not be less than } 0.7C_a I_p W_p \text{ and need not be more than } 4C_a I_p W_p. \quad (5.32-3)$$

Where:

h_x is the element or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0.

h_r is the structure roof elevation with respect to grade.

a_p is the in-structure Component Amplification Factor that varies from 1.0 to 2.5.

A value for a_p shall be selected from Table 5.14. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0.

R_p is the Component Response Modification Factor that shall be taken from Table 5.14, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of non ductile materials, or by use of adhesive, R_p shall equal 1.0.

The design lateral forces determined using Formula (5.32-1) or (5.32-2) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (5.32-1) or (5.32-2) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 5.12.2 or 5.12.3. The Reliability/Redundancy Factor, ρ , may be taken equal to 1.0.

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 5.33.2.4, 5.33.2.8 and 5.33.2.9.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

5.32.3 *Specifying Lateral Forces*

Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

5.32.4 *Relative Motion of Equipment Attachments*

For equipment in Categories 1 and 2 buildings as defined in Table 5.10, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M .

5.32.5 *Alternative Designs*

Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.
2. The force, F_p , and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

5.33 *Detailed Systems Design Requirements*

5.33.1 *General*

All structural framing systems shall comply with the requirements of Section 5.29. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 7 and 8. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 5.33.

All building components in Seismic Zones 2, 3 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads.

In Seismic Zones 2, 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

The structure has plan irregularity Type 5 as given in Table 5.12.

The structure has plan irregularity Type 1 as given in Table 5.12 for both major axes.

A column of a structure forms part of two or more intersecting lateral-force-resisting systems.

Exception: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

5.33.2 *Structural Framing Systems*

5.33.2.1 General. Four types of general building framing systems defined in Section 5.29.6 are recognized in these provisions and shown in Table 5.13. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 7 and 8.

5.33.2.2 Detailing for combinations of systems. For components common to different structural systems, the more restrictive detailing requirements shall be used.

5.33.2.3 Connections. Connections that resist design seismic forces shall be designed and detailed on the drawings.

5.33.2.4 Deformation compatibility. All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, Δ_M , considering $P\Delta$ effects determined in accordance with Section 5.30.9.2 or the deformation induced by a storey drift of 0.0025 times the storey height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

For elements not part of the lateral-force-resisting system, the forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structures and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing.

For concrete and masonry elements that are part of the lateral force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 7.12, Chapter 7.

5.33.2.4.1 Adjoining rigid elements. Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements; provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be

considered when assessing whether a structure shall be designated regular or irregular in Section 5.29.5.1.

5.33.2.4.2 Exterior elements. Exterior nonbearing, non-shear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (5.32-1) or (5.32-2) and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between storeys of not less than two times storey drift caused by wind, the calculated storey drift based on Δ_M or 12 mm (1/2 inch), whichever is greater.
2. Connections to permit movement in the plane of the panel for storey drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.
3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.
4. The body of the connection shall be designed for the force determined by Formula (5.32-2), where $R_p = 3.0$ and $a_p = 1.0$.
5. All fasteners in the connecting system, such as bolts, inserts, welds and dowels, shall be designed for the forces determined by Formula (5.32-2), where $R_p = 1.0$ and $a_p = 1.0$.
6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel.

5.33.2.5 Ties and continuity. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force 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 $0.5 C_a I$ times the weight of the smaller portion.

A positive connection for resisting horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than $0.5 C_a I$ times the dead plus live load.

5.33.2.6 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces. Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (5.33-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 5.12.4.

Exception: In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Formula (5.33-1).

The quantity E_M need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral-force-resisting system. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, Φ , of 1.0.

5.33.2.7 Concrete frames. Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment-resisting frames.
2. In Seismic Zone 2 they shall, as a minimum, be intermediate moment-resisting frames.

5.33.2.8 Anchorage of concrete or masonry walls. Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section and Sections 5.11.4 and 5.32. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 5.33.2.9. Diaphragm deformation shall be considered in the design of the supported walls.

5.33.2.8.1 Out-of-plane wall anchorage to flexible diaphragms. This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 5.30.6, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 5.32 where $R_p = 3.0$ and $a_p = 1.5$.
In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than 6.1 kN per linear meter (420 pounds per linear foot) of wall substituted for E .
See Section 5.11.4 for minimum design forces in other seismic zones.
2. When elements of the wall anchorage system are not loaded concentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.
3. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall be that specified in Section 5.33.2.8.1, Item 1.
4. The strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required by this section.
5. The strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required by this section and these wood elements shall have a minimum actual net thickness of 63.5 mm (2 ½ inches).

5.33.2.9 Diaphragms

1. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.
2. Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (5.33-1)$$

The force F_{px} determined from Formula (5.33-1) need not exceed $1.0 C_a I w_{px}$, but shall not be less than $0.5 C_a I w_{px}$.

- When the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Formula (5.33-1).
3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be obtained using formula (5.33-1) based on the load determined in accordance with section 5.30.2 using R not exceeding 4.
 4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 5.33.2.8. Added chords of sub diaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the wood structural subdiaphragm shall be 2.5:1.
 5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 5.33.2.8. In Seismic Zones 2, 3 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.
 6. Connections of diaphragms to the vertical elements in structures in Seismic Zones 3 and 4, having a plan irregularity of Type 1, 2, 3 or 4 in Table 5.12, shall be designed without considering either the one-third increase or the duration of load increase considered in allowable stresses for elements resisting earthquake forces.
 7. In structures in Seismic Zones 3 and 4 having a plan irregularity of Type 2 in Table 5.12, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:
 - Motion of the projecting wings in the same direction.
 - Motion of the projecting wings in opposing directions.

Exception: This requirement may be deemed satisfied if the procedures of Section 5.31 in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

5.33.2.10 Framing below the base. The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 7 and 8, as appropriate, shall apply to columns supporting discontinuous lateral-force-resisting elements and to SMRF, IMRF, EBF, STMF and MMRWF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

5.33.2.11 Building separation. All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_M . Adjacent buildings on the same property shall be separated by at least Δ_{MT} .

where:

$$MT = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \quad (5.33-2)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement Δ_M of that structure.

Exception: Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

5.34 Nonbuilding Structures

5.34.1 General

5.34.1.1 Scope. Nonbuilding structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 5.34.

5.34.1.2 Criteria. The minimum design seismic forces prescribed in this section are at a level that produces displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reductions in these forces using the coefficient R is permitted where the design of nonbuilding structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, design strengths and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of nonbuilding structures shall use the load combinations or factors specified in Section 5.12.2 or 5.12.3. For nonbuilding structures designed using Section 5.34.3, 5.34.4 or 5.34.5, the Reliability/Redundancy Factor ρ , may be taken as 1.0.

When applicable design strengths and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from approved international standards.

5.34.1.3 Weight W . The weight, W , for nonbuilding structures shall include all dead loads as defined for buildings in Section 5.30.1.1. For purposes of calculating design seismic forces in nonbuilding structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

5.34.1.4 Period. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 5.30.2.2.

5.34.1.5 Drift. The drift limitations of Section 5.30.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. PA effects shall be considered for structures whose calculated drifts exceed the values in Section 5.30.1.3.

5.34.1.6 Interaction effects. In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

5.34.2 Lateral Force

Lateral-force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems which are listed in Table 5.13) shall be selected in accordance with the provisions of Section 5.29.

Exception: Intermediate moment-resisting frames (IMRF) may be used in Seismic Zones 3 and 4 for non building structures in Occupancy Categories 3 and 4 if (1) the structure is less than 15 meters (50 feet) in height and (2) the value R used in reducing calculated member forces and moments does not exceed 2.8.

5.34.3 Rigid Structures

Rigid structures (those with period T less than 0.06 second) and their anchorages shall be designed for the lateral force obtained from Formula (5.34-1).

$$V = 0.7 C_a I W \quad (5.34-1)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

5.34.4 Tanks with Supported Bottoms

Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 5.34 for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

1. A response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.
2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 5.29.4 and 5.29.2, respectively.

5.34.5 Other Nonbuilding Structures

Nonbuilding structures that are not covered by Sections 5.34.3 and 5.34.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 5.30 with the following additions and exceptions:

1. The factors R and Ω_0 shall be as set forth in Table 5.15. The total design base shear determined in accordance with Section 5.30.2 shall not be less than the following:

$$V = 0.56 C_a I W \quad (5.34-2)$$

Additionally, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = (1.6 Z N_v I W)/R \quad (5.34-3)$$

2. The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 5.30.5 or by using the procedures of Section 5.31.

Exception: For irregular structures assigned to Occupancy Categories 1 and 2 that cannot be modeled as a single mass, the procedures of Section 5.31 shall be used.

3. Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of non building structure covered by this section, such a standard may be used, subject to the limitations in this section:

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 5.29.4 and 5.29.2, respectively.

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

5.35 Earthquake-Recording Instrumentations

5.35.1 General

In seismic zones 3 and 4, every building over 10 storeys in height with an aggregate floor area of 9290 square meter ($100,000 \text{ ft}^2$) or more and every building over 15 storeys in height regardless of floor area shall be provided with not less than three approved recording accelerographs. The accelerographs shall be interconnected for common start and common timing.

5.35.2 Location

The instruments will be located in the basement, mid portion and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating "MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT" shall be posted in a conspicuous location.

5.35.3 Maintenance

Maintenance and service of the instruments shall be provided by the owner of the building subject to the approval of the building official. Data produced by the instrument shall be made available to the building official on request.

5.35.4 Instrumentation of Existing Buildings

All owners of existing structures selected by the jurisdiction authorities shall provide accessible space for the installation of appropriate strong motion recording instruments. Location of said instruments shall be determined by the jurisdiction authorities. The jurisdiction authorities shall make arrangements to provide, maintain and service the instruments. Data shall be the property of the jurisdiction authorities but the copies of individual records shall be made available to the public on request and payment of an appropriate fee.

Table 5.1 – Uniform and Concentrated Loads

Use or occupancy		Uniform Load¹		Concentrated Load	
Category	Description	kN/m²	psf	kN	lb
1. Access floor system	Office use	2.4	50	9.0	2,000 ²
	Computer use	4.8	100	9.0	2,000 ²
2. Armories		7.2	150	0	0
3. Assembly areas ³ and auditoriums and balconies therewith	Fixed seating areas	2.4	50	0	0
	Movable seating and other areas	4.8	100	0	0
	Stage areas and enclosed platforms	6.0	125	0	0
4. Cornices and marquees		2.9	60 ⁴	0	0
5. Exit facilities ⁵		4.8	100	0	0 ⁶
6. Garages	General storage and/or repair	4.8	100		See Footnote ⁷
	Private or pleasure-type motor vehicle storage	2.4	50		See Footnote ⁷
7. Hospitals	Wards and rooms	1.9	40	4.5	1,000 ²
8. Libraries	Reading rooms	2.9	60	4.5	1,000 ²
	Stack rooms	6.0	125	6.7	1,500 ²
9. Manufacturing	Light	3.6	75	9.0	2,000 ²
	Heavy	6.0	125	13.5	3,000 ²
10. Offices		2.4	50	9.0	2,000 ²
11. Printing plants	Press rooms	7.2	150	11.2	2,500 ²
	Composing and linotype rooms	4.8	100	9.0	2,000 ²
12. Residential ⁸	Basic floor area	1.9	40	0	0 ⁶
	Exterior balconies	2.9	60 ⁴	0	0
	Decks	1.9	40 ⁴	0	0
	Storage	1.9	40	0	0
13. Restrooms ⁹					
14. Reviewing stands, grandstands, bleachers, and folding and telescoping seating		4.8	100	0	0
15. Roof decks	Same as area served or for the type of occupancy accommodated				
16. Schools	Classrooms	1.9	40	4.5	1,000 ²
17. Sidewalks and driveways	Public access	12.0	250		See Footnote ⁷
18. Storage	Light	6.0	125		
	Heavy	12.0	250		
19. Stores		4.8	100	13.5	3,000 ²
20. Pedestrian bridges and walkways		4.8	100		

¹See Section 5.7 for live load reductions.²See Section 5.7.3.3, first paragraph, for area of load application.³Assembly areas include such occupancies as dance halls, drill rooms, gymnasiums, playgrounds, plazas, terraces and similar occupancies that are generally accessible to the public.⁴When snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design as determined by the building official. See Section 5.14. For special purpose roofs, see Section 5.7.4.4.⁵Exit facilities shall include such uses as corridors serving an occupant load of 10 or more persons, exterior exit balconies, stairways, fire escapes and similar uses.⁶Individual stair treads shall be designed to support a 1.33 kN (300 lbs) concentrated load placed in a position that would cause maximum stress. Stair stringers may be designed for the uniform load set forth in the table.⁷See Section 5.7.3.3, second paragraph, for concentrated loads. See Table 5.2 for vehicle barriers.⁸Residential occupancies include private dwellings, apartments and hotel guest rooms.⁹Restroom loads shall not be less than the load for the occupancy with which they are associated, but need not exceed 2.4 kN/m² (50 psf).

Table 5.2 – Special Loads¹

Use		Vertical Load		Lateral Load	
Category	Description	kN/m ²	psf	kN/m ²	psf
1. Construction, public access at site (live load)	Walkways	7.2	150		
	Canopy	7.2	150		
2. Grandstands, reviewing stands, bleachers, and folding and telescoping seating (live load)	Seats and footboards	5.8	120 ²	See Footnote ³	
3. Stage accessories (live load)	Catwalks	1.2	40		
	Followspot, projection and control rooms	2.4	50		
4. Ceiling framing (live load)	Over stages	1.0	20		
	All uses except over stages	0.5	10 ⁴		
5. Partitions and interior walls, see Sec 5.11.5 (live load)				0.3	5
6. Elevators and dumbwaiters (dead and live loads)		2 x total loads ⁵			
7. Mechanical and electrical equipment (dead load)		Total loads			
8. Cranes (dead and live loads)	Total load including impact increase	1.25 x total load ⁶		0.10 x total load ⁷	
9. Balcony railings and guardrails	Exit facilities serving an occupant load greater than 2.4 kN/m ² (50 psf)			2.4	50 ⁸
	Other than exit facilities			1.0	20 ⁸
	Components			1.2	25 ⁹
10. Vehicle barriers	See Footnote ¹⁴			287.4	6,000 ¹⁰
11. Handrails		See Footnote ¹¹		See Footnote ¹¹	
12. Storage racks	Over 2.438 m (8 feet) high	Total loads ¹²		See Table 5.14	
13. Fire sprinkler structural support		1112 N plus weight of water-filled pipe ¹³	250 lbs plus weight of water-filled pipe ¹³	See Table 5.14	
14. Explosion exposure	Hazardous occupancies, see Section 307.10 (UBC 1997)				

¹The tabulated loads are minimum loads. Where other vertical loads required by this code or required by the design would cause greater stresses, they shall be used.

²Newton per linear meter (x 0.0685 for lb/ft).

³Lateral sway bracing loads of 350 N/m (24 lb/ft) parallel and 146 N/m (10 lb/ft) perpendicular to seat and footboards.

⁴Does not apply to ceilings that have sufficient total access from below, such that access is not required within the space above the ceiling. Does not apply to ceilings if the attic areas above the ceiling are not provided with access. This live load need not be considered as acting simultaneously with other live loads imposed upon the ceiling framing or its supporting structure.

⁵Where Appendix Chapter 30 of UBC-1997 has been adopted, see reference standard cited therein for additional design requirements.

⁶The impact factors included are for cranes with steel wheels riding on steel rails. They may be modified if substantiating technical data acceptable to the building official is submitted. Live loads on crane support girders and their connections shall be taken as the maximum crane wheel loads. For pendant-operated traveling crane support girders and their connections, the impact factors shall be 1.10.

⁷This applies in the direction parallel to the runway rails (longitudinal). The factor for forces perpendicular to the rail is 0.20 x the transverse traveling loads (trolley, cab, hooks and lifted loads). Forces shall be applied at top of rail and may be distributed among rails of multiple rails cranes and shall be distributed with due regard for lateral stiffness of the structures supporting these rails.

⁸A load in Newton per linear meter (x 0.0685 for lb/ft) to be applied horizontally at right angles to the top rail.

⁹Intermediate rails, panel fillers and their connections shall be capable of withstanding a load of 1.2 kN/m² (25 psf) applied horizontally at right angles over the entire tributary area, including openings and spaces between rails. Reactions due to this loading need not be combined with those of Foot-note 8.

¹⁰A horizontal load in Newton (lb) applied at right angles to the vehicle barrier at a height of 457 mm (18 inches) above the parking surface. The force may be distributed over a 305 millimeter-square (1-foot-square) area.

¹¹The mounting of handrails shall be such that the completed handrail and supporting structure are capable of withstanding a load of at least 890 N (200 lbs) applied in any direction at any point on the rail. These loads shall not be assumed to act cumulatively with Item 9.

¹²Vertical members of storage racks shall be protected from impact forces of operating equipment, or racks shall be designed so that failure of one vertical member will not cause collapse of more than the bay or bays directly supported by that member.

¹³The 1.11 kN (250-pound) load is to be applied to any single fire sprinkler support point but not simultaneously to all support joints.

¹⁴In parking areas where any parking area is located more than 1.5 m (5 feet) above the adjacent grade, vehicle barriers shall be provided.

Table 5.3 – Minimum Roof Live Loads¹

Roof Slope	Method 1			Method 2		
	Tributary Loaded Area in square meter (square feet) for Any Structural Member			Uniform load kN/m ² (psf)	Rate of Reduction <i>r</i> (percentage)	Maximum Reduction <i>R</i> (percentage)
	0 to 20 (0 to 200)	21 to 60 (201 to 600)	Over 60 (600)			
	Uniform Load kN/m ² (psf)					
1. Flat ³ or rise less than 4 units vertical in 12 units horizontal (33.3% slope). Arch or dome with rise less than one eighth of span.	1 (20)	0.8 (16)	0.6 (12)	1 (20)	.08	40
2. Rise 4 units vertical to less than 12 units vertical in 12 units horizontal (33% to less than 100% slope). Arch or dome with rise one eighth of span to less than three eighths of span.	0.8 (16)	0.7 (14)	0.6 (12)	0.8 (16)	.06	25
3. Rise 12 units vertical in 12 units horizontal (100% slope) and greater. Arch or dome with rise three eights or span or greater.	0.6 (12)	0.6 (12)	0.6 (12)	0.6 (12)	No reductions permitted	
4. Awnings except cloth covered.	0.3 (5)	0.3 (5)	0.3 (5)	0.3 (5)		
5. Greenhouses, lath houses and agricultural buildings ⁴ .	0.5 (10)	0.5 (10)	0.5 (10)	0.5 (10)		

¹Where snow loads occur, the roof structure shall be designed for such loads as determined by the building official. See Section 5.14. For special-purpose roofs, see Section 5.7.4.4.

²See Sections 5.7.5 and 5.7.6 for live load reductions. The rate of reduction *r* in Section 5.7.5 Formula (5.7-1) shall be as indicated in the table. The maximum reduction *R* shall not exceed the value indicated in the table.

³A flat roof is any roof with a slope of less than $\frac{1}{4}$ unit in vertical 12 units horizontal (2% slope). The live load for flat roofs is in addition to the ponding load required by Sections 5.11.7.

⁴See Section 5.7.4.4 for concentrated load requirements for greenhouse roof members.

Table 5.4 Maximum Allowable Deflection for Structural Members¹

Type of Member	Member Loaded With Live Load Only (<i>L</i>)	Member Loaded With Live Load Plus Dead Load (<i>L + K.D.</i>)
Roof member supporting plaster or floor member	<i>I</i> /360	<i>I</i> /240

¹Sufficient slope or camber shall be provided for flat roofs in accordance with Section 5.11.7

L. - live load.

D. - dead load.

K. - factor as determined by Table 5.5.

I - length of member in same units as deflection.

Table 5.5-Value of “K”

Reinforced Concrete ¹	Steel
<i>T/(1+50ρ')</i>	0

¹See also Section 1909 (UBC 1997) for definitions and other requirements.

ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. Time-dependent factor *T* for sustained loads may be taken equal to:

five years or more	2.0
twelve months	1.2
six months	1.4
three months	1.0

Table 5.6 –Wind Stagnation Pressure (q_s) at Standard Height of 10 Meters (33 Feet)

Basic wind speed (km/h)¹ (x 0.621 for mph)	113	129	145	161	177	193	209
Pressure q_s (kN/m²) (x 20.9 for psf)	0.60	0.78	1.0	1.22	1.48	1.77	2.07

¹ wind speed from Section 5.18

Table 5.7-Combined Height, Exposure and Gust Factor Coefficient (C_e)¹

Height Above Average Level of Adjoining Ground (Meter) x 3.28 for feet	Exposure D	Exposure C	Exposure B
0-4.600	1.39	1.06	0.62
6.000	1.45	1.13	0.67
7.620	1.50	1.19	0.72
9.150	1.54	1.23	0.76
12.192	1.62	1.31	0.84
18.300	1.73	1.43	0.95
24.380	1.81	1.53	1.04
30.480	1.88	1.61	1.13
36.600	1.93	1.67	1.20
48.770	2.02	1.79	1.31
61.000	2.10	1.87	1.42
91.440	2.23	2.05	1.63
122.000	2.34	2.19	1.80

¹ Values for intermediate heights above 4.6 m (15 feet) may be interpolated.

Table 5.8 – Pressure Coefficients (C_q)

Structure or Part Thereof	Description	C_q Factor																																							
1. Primary frames and systems	Method 1 (Normal force method) <table> <tr> <td>Walls:</td> <td>Windward wall</td> <td>0.8 inward</td> </tr> <tr> <td></td> <td>Leeward wall</td> <td>0.5 outward</td> </tr> <tr> <td>Roofs¹:</td> <td>Wind perpendicular to ridge</td> <td></td> </tr> <tr> <td></td> <td>Leeward roof or flat roof</td> <td>0.7 outward</td> </tr> <tr> <td></td> <td>Windward roof</td> <td></td> </tr> <tr> <td></td> <td>Less than 2:12 (16.7%)</td> <td>0.7 outward</td> </tr> <tr> <td></td> <td>Slope 2:12 (16.7%) to less than 9:12 (75%)</td> <td>0.9 outward or 0.3 inward</td> </tr> <tr> <td></td> <td>Slope 9:12 (75%) to 12:12 (100%)</td> <td>0.4 inward</td> </tr> <tr> <td></td> <td>Slope > 12:12 (100%)</td> <td>0.7 inward</td> </tr> <tr> <td></td> <td>Wind parallel to ridge and flat roofs</td> <td>0.7 outward</td> </tr> </table> Method 2 (Projected area method) <table> <tr> <td>On vertical projected area</td> <td>Structures 12 192 mm (40 feet) or less in height</td> <td>1.3 horizontal any direction</td> </tr> <tr> <td></td> <td>Structures over 12 192 mm (40 feet) in height</td> <td>1.4 horizontal any direction</td> </tr> <tr> <td></td> <td>On horizontal projected area¹</td> <td>0.7 upward</td> </tr> </table>	Walls:	Windward wall	0.8 inward		Leeward wall	0.5 outward	Roofs ¹ :	Wind perpendicular to ridge			Leeward roof or flat roof	0.7 outward		Windward roof			Less than 2:12 (16.7%)	0.7 outward		Slope 2:12 (16.7%) to less than 9:12 (75%)	0.9 outward or 0.3 inward		Slope 9:12 (75%) to 12:12 (100%)	0.4 inward		Slope > 12:12 (100%)	0.7 inward		Wind parallel to ridge and flat roofs	0.7 outward	On vertical projected area	Structures 12 192 mm (40 feet) or less in height	1.3 horizontal any direction		Structures over 12 192 mm (40 feet) in height	1.4 horizontal any direction		On horizontal projected area ¹	0.7 upward	
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	On horizontal projected area ¹	0.7 upward																																							
2. Elements and components not in areas of discontinuity ²	<table> <tr> <td>Wall elements</td> <td>All structures</td> <td>1.2 inward</td> </tr> <tr> <td></td> <td>Enclosed and unenclosed structures</td> <td>1.2 outward</td> </tr> <tr> <td></td> <td>Partially enclosed structures</td> <td>1.6 outward</td> </tr> <tr> <td></td> <td>Parapets walls</td> <td>1.3 inward or outward</td> </tr> </table> <table> <tr> <td>Roof elements³</td> <td>Enclosed and unenclosed structures</td> <td>1.3 outward</td> </tr> <tr> <td></td> <td>Slope <7:12 (58.3%)</td> <td>1.3 outward or inward</td> </tr> <tr> <td></td> <td>Slope 7:12 (58.3%) to 12:12 (100%)</td> <td></td> </tr> <tr> <td></td> <td>Partially enclosed structures</td> <td></td> </tr> <tr> <td></td> <td>Slope <2:12 (16.7%)</td> <td>1.7 outward</td> </tr> <tr> <td></td> <td>Slope 2:12 (16.7%) to 7:12 (58.3%)</td> <td>1.6 outward or 0.8 inward</td> </tr> <tr> <td></td> <td>Slope > 7:12 (58.3%) to 12:12 (100%)</td> <td>1.7 outward or inward</td> </tr> </table>	Wall elements	All structures	1.2 inward		Enclosed and unenclosed structures	1.2 outward		Partially enclosed structures	1.6 outward		Parapets walls	1.3 inward or outward	Roof elements ³	Enclosed and unenclosed structures	1.3 outward		Slope <7:12 (58.3%)	1.3 outward or inward		Slope 7:12 (58.3%) to 12:12 (100%)			Partially enclosed structures			Slope <2:12 (16.7%)	1.7 outward		Slope 2:12 (16.7%) to 7:12 (58.3%)	1.6 outward or 0.8 inward		Slope > 7:12 (58.3%) to 12:12 (100%)	1.7 outward or inward							
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3. Elements and components in areas of discontinuities ^{2,4,5}	<table> <tr> <td>Wall corners⁶</td> <td>Roof eaves, rakes or ridges without overhangs⁶</td> <td>1.5 outward or 1.2 inward</td> </tr> <tr> <td></td> <td>Slope <2:12 (16.7%)</td> <td>2.3 upward</td> </tr> <tr> <td></td> <td>Slope 2:12 (16.7%) to 7:12 (58.3%)</td> <td>2.6 outward</td> </tr> <tr> <td></td> <td>Slope >7:12 (58.3%) to 12:12 (100%)</td> <td>1.6 outward</td> </tr> <tr> <td></td> <td>For slopes less than 2:12 (16.7%)</td> <td></td> </tr> <tr> <td></td> <td>Overhangs at roof eaves, rakes or ridges, and canopies</td> <td>0.5 added to values above</td> </tr> </table>	Wall corners ⁶	Roof eaves, rakes or ridges without overhangs ⁶	1.5 outward or 1.2 inward		Slope <2:12 (16.7%)	2.3 upward		Slope 2:12 (16.7%) to 7:12 (58.3%)	2.6 outward		Slope >7:12 (58.3%) to 12:12 (100%)	1.6 outward		For slopes less than 2:12 (16.7%)			Overhangs at roof eaves, rakes or ridges, and canopies	0.5 added to values above																						
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	Overhangs at roof eaves, rakes or ridges, and canopies	0.5 added to values above																																							
4. Chimneys, tanks and solid towers	<table> <tr> <td>Square or rectangular</td> <td>1.4 any direction</td> </tr> <tr> <td>Hexagonal or octagonal</td> <td>1.1 any direction</td> </tr> <tr> <td>Round or elliptical</td> <td>0.8 any direction</td> </tr> </table>	Square or rectangular	1.4 any direction	Hexagonal or octagonal	1.1 any direction	Round or elliptical	0.8 any direction																																		
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5. Open-frame towers ^{7,8}	<table> <tr> <td>Square and rectangular</td> <td></td> </tr> <tr> <td>Diagonal</td> <td>4.0</td> </tr> <tr> <td>Normal</td> <td>3.6</td> </tr> <tr> <td>Triangular</td> <td>3.2</td> </tr> </table>	Square and rectangular		Diagonal	4.0	Normal	3.6	Triangular	3.2																																
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Triangular	3.2																																								
6. Tower accessories (such as ladders, conduit, lights and elevators)	<table> <tr> <td>Cylindrical members</td> <td></td> </tr> <tr> <td>51 mm (2 inches) or less in diameter</td> <td>1.0</td> </tr> <tr> <td>Over 51 mm (2 inches) in diameter</td> <td>0.8</td> </tr> <tr> <td>Flat or angular members</td> <td>1.3</td> </tr> </table>	Cylindrical members		51 mm (2 inches) or less in diameter	1.0	Over 51 mm (2 inches) in diameter	0.8	Flat or angular members	1.3																																
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7. Signs, flagpoles, light poles, minor structures ⁸		1.4 any direction																																							

¹For one storey or the top storey of multistorey partially enclosed structures, an additional value of 0.5 shall be added to the outward C_q . The most critical combination shall be used for design. For definition of partially enclosed structures, See Section 5.16.

² C_q values listed are for 0.93 m² (10 ft²) tributary areas. For tributary areas of 9.29 m² (100 ft²), the value of 0.3 may be subtracted from C_q , except for areas at discontinuities with slopes less than 7 units vertical in 12 units horizontal (58.3% slope) where the value of 0.8 may be subtracted from C_q . Interpolation may be used for tributary areas between 0.93 and 9.29 m² (10 and 100 ft²). For tributary areas greater than 92.9 m² (1,000 ft²), use primary frame values.

³For slopes greater than 12 units vertical in 12 units horizontal (100% slope), use wall element values.

⁴Local pressures shall apply over a distance from the discontinuity of 3048 mm (10 feet) or 0.1 times the least width of the structure, whichever is smaller.

⁵Discontinuities at wall corners or roof ridges are defined as discontinuous breaks in the surface where the included interior angle measures 170 degrees or less.

⁶Load is to be applied on either side of discontinuity but not simultaneously on both sides.

⁷Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.

⁸Factors for cylindrical elements are two thirds of those for flat or angular elements.

Table 5.9 – Seismic Zone Factor Z

Zone	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

Note: The zone shall be determined from the seismic zone map in Fig. 2.1 or from Table 2.2.

Table 5.10 – Occupancy Category⁴

Occupancy Category	Occupancy or Functions of Structure	Seismic Importance Factor, I	Seismic Importance¹ Factor, I_p	Wind Importance Factor, I_w
1. Essential facilities ²	Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	1.25	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1,2,6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1,2 or 7 Occupancy.	1.25	1.50	1.15
3. Special occupancy structures ³	Group A, Divisions 1,2 and 2.1 Occupancies Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation.	1.00	1.00	1.00
4. Standard occupancy structures ³	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

¹The limitation of I_p for panel connections in Section 5.33.2.4 shall be 1.0 for the entire connector.

²Structural observation requirements are given in Chapter 6, Section 6.2.

³For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

⁴See Table 5.21 for description of occupancies by Group and Division.

Table 5.11 – Vertical Structural Irregularities

Irregularity Type and Definition	Reference Section
1. Stiffness irregularity – soft storey A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average stiffness of the three storeys above.	5.29.8.4, Item 2
2. Weight (mass) irregularity Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150 percent of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered.	5.29.8.4, Item 2
3. Vertical geometric irregularity Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any storey is more than 130 percent of that in an adjacent storey. One-storey penthouses need not be considered.	5.29.8.4, Item 2
4. In-plane discontinuity in vertical lateral-force-resisting element An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.	5.30.8.2
5. Discontinuity in capacity – weak storey A weak storey is one in which the storey strength is less than 80 percent of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey shear for the direction under consideration.	5.29.9.1

Table 5.12 – Plan Structural Irregularities

Irregularity Type and Definition	Reference Section
1. Torsional irregularity – to be considered when diaphragms are not flexible Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the storey drifts of the two ends of the structure.	5.33.1; 5.33.2.9, Item 6
2. Re-entrant corners Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	5.33.2.9, Items 6 and 7
3. Diaphragm discontinuity Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.	5.33.2.9, Item 6
4. Out-of-plane offsets Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	5.30.8.2, 5.33.2..9, Item 6; 2213.8 of UBC97
5. Nonparallel systems The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	5.33.1

Table 5.13 – Structural Systems¹

Basic Structural System²	Lateral-Force-Resisting System Description	R	Ω_0	Height Limit for Seismic Zones 3 and 4	
				(m)	(ft)
1. Bearing wall system	1. Light-framed walls with shear panels	5.5	2.8	20	65
	a. Wood structural panel walls for structures three storeys or less	4.5	2.8	20	65
	b. All other light-framed walls				
	2. Shear walls	4.5	2.8	50	160
	a. Concrete	4.5	2.8	50	160
	b. Masonry				
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	20	65
	4. Braced frames where bracing carries gravity load	4.4	2.2	50	160
	a. Steel	2.8	2.2	-	-
	b. Concrete ³	2.8	2.2	20	65
2. Building frame system	c. Heavy timber				
	1. Steel eccentrically braced frame (EBF)	7.0	2.8	75	240
	2. Light-framed walls with shear panels	6.5	2.8	20	65
	a. Wood structural panel walls for structures three storeys or less	5.0	2.8	20	65
	b. All other light-framed walls				
	3. Shear walls	5.5	2.8	75	240
	a. Concrete	5.5	2.8	50	160
	b. Masonry				
	4. Ordinary braced frames	5.6	2.2	50	160
	a. Steel	5.6	2.2	-	-
3. Moment-resisting frame system	b. Concrete ³	5.6	2.2	20	65
	c. Heavy timber				
	5. Special concentrically braced frames	6.4	2.2	75	240
	a. Steel				
	1. Special moment-resisting frame (SMRF)	8.5	2.8	N.L.	N.L.
	a. Steel	8.5	2.8	N.L.	N.L.
	b. Concrete ⁴				
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	50	160
	3. Concrete intermediate moment-resisting frame (IMRF) ⁵	5.5	2.8	-	-
	4. Ordinary moment-resisting frame (OMRF)	4.5	2.8	50	160
4. Dual systems	a. Steel ⁶	3.5	2.8	-	-
	b. Concrete ⁷	6.5	2.8	75	240
	5. Special truss moment frames of steel (STMF)				
	1. Shear walls	8.5	2.8	N.L.	N.L.
	a. Concrete with SMRF	4.2	2.8	50	160
	b. Concrete with steel OMRF	6.5	2.8	50	160
	c. Concrete with concrete IMRF ⁵	5.5	2.8	50	160
	d. Masonry with SMRF	4.2	2.8	50	160
	e. Masonry with steel OMRF	4.2	2.8	-	-
	f. Masonry with concrete IMRF ³	6.0	2.8	50	160
5. Cantilevered column building systems	g. Masonry with masonry MMRWF				
	2. Steel EBF	8.5	2.8	N.L.	N.L.
	a. With steel SMRF	4.2	2.8	50	160
	b. With steel OMRF				
	3. Ordinary braced frames	6.5	2.8	N.L.	N.L.
	a. Steel with steel SMRF	4.2	2.8	50	160
	b. Steel with Steel OMRF	6.5	2.8	-	-
	c. Concrete with concrete SMRF ³	4.2	2.8	-	-
	d. Concrete with concrete IMRF ³				
	4. Special concentrically braced frames	7.5	2.8	N.L.	N.L.
6. Shear wall-frame interaction systems	a. Steel with steel SMRF	4.2	2.8	50	160
	b. Steel with steel OMRF				
7. Undefined systems	See Sections 5.29.6.7 and 5.29.9.2	-	-	-	-

N.L. – no limit

¹See section 5.30.4 for combination of structural systems.²Basic structural system are defined in Section 5.29.6.³Prohibited in Seismic Zones 3 and 4.⁴Includes precast concrete conforming to Section 1921.2.7 (UBC 1997).⁵Prohibited in Seismic Zones 3 and 4, except as permitted in Section 5.34.2.⁶Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Chapter 8 may use a R value of 8.⁷Total height of the building including cantilevered columns.⁸Prohibited in Seismic Zones 2A, 2B, 3 and 4. See section 5.33.2.7.

Table 5.14 – Horizontal Force Factors, a_p and R_p

Elements of Structures and Nonstructural Components and Equipments¹	a_p	R_p	Footnote
1. Elements of Structures			
A. Walls including the following			
(1) Unbraced (cantilevered) parapets.	2.5	3.0	
(2) Exterior wall at or above the ground floor and parapets braced above their centers of gravity.	1.0	3.0	2
(3) All interior-bearing and nonbearing walls.	1.0	3.0	2
B. Penthouse (except when framed by an extension of the structural frame).	2.5	4.0	
C. Connections for prefabricated structural elements other than walls. See also section 5.32.2.	1.0	3.0	3
2. Nonstructural Components			
A. Exterior and interior ornamentations and appendages.	2.5	3.0	
B. Chimneys, stacks and trussed towers supported on or projecting above the roof:			
(1) Laterally braced or anchored to the structural frame at a point below their centers of mass.	2.5	3.0	
(2) Laterally braced or anchored to the structural frame at or above their centers of mass.	1.0	3.0	
C. Signs and billboards.	2.5	3.0	
D. Storage racks (include contents) over 1829 mm (6 feet) tall.	2.5	4.0	4
E. Permanent floor-supported cabinets and book stacks more than 1829 mm (6 feet) in height (include contents).	1.0	3.0	5
F. Anchorage and lateral bracing for suspended ceilings and light fixtures.	1.0	3.0	3,6,7,8
G. Access floor systems.	1.0	3.0	
H. Masonry or concrete fences over 1829 mm (6 feet) high.	1.0	3.0	
I. Partitions.	1.0	3.0	
3. Equipment			
A. Tanks and vessels (include contents), including support systems.	1.0	3.0	
B. Electrical, mechanical and plumbing equipment and associated conduit and ductwork and piping.	1.0	3.0	5,10,11,12,13, 14,15,16
C. Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass.	2.5	3.0	5,10,14,15,16
D. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support for battery racks and fuel tanks necessary for operations of emergency equipment. See also section 5.32.2.	1.0	3.0	17,18
E. Temporary containers with flammable or hazardous materials.	1.0	3.0	19
4. Other Components			
A. Rigid components with ductile material and attachments.	1.0	3.0	1
B. Rigid components with nonductile material or attachments.	1.0	1.5	1
C. Flexible components with ductile material and attachments	2.5	3.0	1
D. Flexible components with nonductile material or attachments.	2.5	1.5	1

¹See Section 5.27 for definitions of flexible components and rigid components.²See sections 5.33.2.4 and 5.33.2.8 for concrete and masonry walls and section 5.32.2 for connections for panel connectors for panels.³Applies to Seismic Zones 2, 3 and 4 only.⁴Ground supported steel storage racks may be designed using the provisions of Sections 5.34. Chapter 8 may be used for design, provided seismic design forces are equal to or greater than those specified in Section 5.32.2 or 5.34.2 as appropriate.⁵Only anchorage or restraints need be designed.⁶Ceiling weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, ceiling weight of not less than 0.20 kN/m² (4 psf) shall be used.⁷Ceiling constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed, provided the walls are not over 15.24 m (50 feet) apart.⁸Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structures above.⁹ W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 0.50 kN/m² (10 psf) partition load allowance.¹⁰Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers and life-safety equipment. It shall include major conduit ducting and piping which services such machinery and equipment and fire sprinkler systems. See section 5.32.2 for additional requirements for determining a_p for nonrigid or flexibly mounted equipment.¹¹Seismic restraints may be omitted from piping and duct supports if all the following conditions are satisfied:^{11.1} Lateral motion of the piping or duct will not cause damaging impact with other systems.^{11.2} The piping or duct is made of ductile material with ductile connections.^{11.3} Lateral motion of the piping or duct does not cause impact of fragile appurtenances (e.g. sprinkler heads) with any other equipment, piping or structural member.^{11.4} Lateral motion of the piping or duct does not cause loss of system vertical support.

^{11.5} Rod-hung supports of less than 305 mm (12 inches) in length have top connections that cannot develop moments.

^{11.6} Support members cantilevered up from the floor are checked for stability.

¹² Seismic restraints may be omitted from electrical raceways, such as cable trays, conduit and bus ducts, if all the following conditions are satisfied:

^{12.1} Lateral motion of the raceway will not cause damaging impact with other systems.

^{12.2} Lateral motion of the raceway does not cause loss of system vertical support.

^{12.3} Rod-hung supports of less than 305 mm (12 inches) in length have top connections that cannot develop moments.

^{12.4} Support members cantilevered up from the floor are checked for stability.

¹³ Piping, ducts and electrical raceways, which must be functional following an earthquake, spanning between different buildings or structural systems shall be sufficiently flexible to withstand relative motion of support points assuming out of phase motion.

¹⁴ Vibration isolators supporting equipment shall be designed for lateral loads or restrained from displacing laterally by other means. Restraint shall also be provided, which limits vertical displacement, such that lateral restraints do not become disengaged. a_p and R_p for equipment supported on vibration isolators shall be taken 2.5 and 1.5 respectively, except that if the isolation mounting frame is supported by shallow or expansion anchors, the design forces for the anchors calculated by Formula (5.32-1), (5.32-2) or (5.32-3) shall be additionally multiplied by a factor of 2.0.

¹⁵ Equipment anchorage shall not be designed such that lateral loads are resisted by gravity friction (e.g., friction clips).

¹⁶ Expansion anchors, which are required to resist seismic loads in tension, shall not be used where operational vibrating loads are present.

¹⁷ Movement of components within electrical cabinets, rack and skid-mounted equipment and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, shall be restricted by attachment to anchored equipment or support frames.

¹⁸ Batteries on racks shall be restrained against movement in all directions due to earthquake forces.

¹⁹ Seismic restraints may include straps, chains, bolts, barriers or other mechanisms that prevent sliding, falling and breach of containment of flammable and toxic materials. Friction forces may not be used to resist lateral loads in these restraints unless positive uplift restraint is provided which ensures that the friction forces act continuously.

Table 5.15 – R and Ω_o Factors for Non-Building Structures

Structure Type	R	Ω_o
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	2.9	2.0
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	2.2	2.0
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks.	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11 All other self-supporting structures not otherwise covered.	2.9	2.0

Table 5.16 – Seismic Coefficients C_a

Soil Profile Type ²	Seismic Zone Factor, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

¹ Site Specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F .

²For Soil Profile Types, See Table 4.1.

Table 5.17 – Seismic Coefficient C_v

Soil Profile Type ²	Seismic Zone Factor, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

¹ Site Specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F .

²For Soil Profile Types, See Table 4.1.

Table 5.18 – Near Source Factor N_a ¹

Seismic Source Type	Closest Distance to Known Seismic Source ^{2,3}		
	$\leq 2 \text{ km}$	5 km	$\geq 10 \text{ km}$
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

¹ The Near Source Factor may be based on the linear interpolation of values for distance other than those shown in the table.

² The location and type of seismic sources to be used for design shall be established based on approved geotechnical data.

³ The closet distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all source shall be used for design.

Table 5.19 – Near Source Factor N_v ¹

Seismic Source Type	Closest Distance to Known Seismic Source ^{2,3}			
	$\leq 2 \text{ km}$	5 km	10 km	$\geq 15 \text{ km}$
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

¹ The Near Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

² The location and type of seismic sources to be used for design shall be established based on approved geotechnical data.

³ The closet distance to seismic source shall be taken as the minimum distance between the site and the areas described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

Table 5.20 – Seismic Source Type¹

Seismic Source Type	Seismic Source Description	Seismic Source Definition ²	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

¹ Subduction sources shall be evaluated on a site-specific basis.

² Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

Table 5.21– Description of Occupancies by Group and Division

Group and Division	Section of UBC 1997	Description of Occupancy
A-1	303.1.1	A building or portion of a building having an assembly room with an occupant load of 1,000 or more and legitimate stage.
A-2		A building or portion of a building having an assembly room with an occupant load of less than 1,000 and a legitimate stage.
A-2.1		A building or portion of a building having an assembly room with an occupant load of 300 or more without a legitimate stage, including such buildings used for educational purposes and not classed as a Group E or Group B Occupancy.
A-3		Any building or portion of a building having an assembly room with an occupant load of less than 300 without a legitimate stage, including such buildings used for educational purposes and not classed as a Group E or Group B Occupancy.
A-4		Stadiums, reviewing stands and amusement park structures not included within other Group A Occupancies.
B	304.1	A building or structure, or a portion thereof, for office, professional or service-type transactions, including storage of records and accounts; eating and drinking establishments with an occupant load of less than 50.
E-1	305.1	Any building used for educational purposes through the 12 th grade by 50 or more persons for more than 12 hours per week or four hours in any one day.
E-2		Any building used for educational purposes through the 12 th grade by less than 50 persons for more than 12 hours per week or four hours in any one day.
E-3		Any building or portion thereof used for day-care purposes for more than six persons.
F-1	306.1	Moderate-hazard factory and industrial occupancies include factory and industrial uses not classified as Group F, Division 2 Occupancies.
F-2		Low-hazard factory and industrial occupancies include facilities producing noncombustible or non-explosive materials that during finishing, packing or processing do not involve a significant fire hazard.
H-1	307.1	Occupancies with a quantity of material in the building in excess of those listed in Table 3-D that present a high explosion hazard as listed in Section 307.1.1.
H-2		Occupancies with a quantity of material in the building in excess of those listed in Table 3-D that present a moderate explosion hazard or hazard from accelerated burning as listed in Section 307.1.1.
H-3		Occupancies with a quantity of material in the building in excess of those listed in Table 3-D that present a high fire or physical hazard as listed in Section 307.1.1.
H-4		Repair garages not classified as Group S, Division 3 Occupancies.
H-5		Aircraft repair hangars not classified as Group S, Division 5 Occupancies.
H-6	307.1 and 307.11	Semiconductor fabrication facilities and comparable research and development areas when the facilities in which hazardous production materials are used, and the aggregate quantity of material is in excess of those listed in Table 3-D or 3-E.
H-7	307.1	Occupancies having quantities of materials in excess of those listed in Table 3-E that are health hazards as listed in Section 307.1.1
I-1.1	308.1	Nurseries for the full-time care of children under the age of six (each accommodating more than five children), hospitals, sanitariums, nursing homes with non-ambulatory patients and similar buildings (each accommodating more than five patients).
I-1.2		Health-care centers for ambulatory patients receiving outpatient medical care which may render the patient incapable of unassisted self-preservation (each tenant space accommodating more than five such patients).
I-2		Nursing homes for ambulatory patients, homes for children six years of age or over (each accommodating more than five persons).
I-3		Mental hospitals, mental sanitariums, jails, prisons, reformatories and buildings where personal liberties of inmates are similarly restrained.
M	309.1	A building or structure, or a portion thereof, for the display and sale of merchandise, and involving stocks of goods, wares or merchandise, incidental to such purposes and accessible to the public.
R-1	310.1	Hotels and apartment houses, congregate residences (each accommodating more than 10 persons).
R-3		Dwellings, lodging houses, congregate residences (each accommodating 10 or fewer persons).

S-1	311.1	Moderate hazard storage occupancies including buildings or portions of buildings used for storage of combustible materials not classified as Group S, Division 2 or Group H Occupancies.
S-2		Low-hazard storage occupancies including buildings or portions of buildings used for storage of non-combustible materials.
S-3		Repair garages where work is limited to exchange of parts and maintenance not requiring open flame or welding, and parking garages not classified as Group S, Division 4 Occupancies.
S-4		<u>Open parking garages.</u>
S-5		Aircraft hangers and helistops.
U-1	312.1	Private garages, carports, sheds and agricultural buildings.
U-2		Fences over 1.83 m (6 feet) high, tanks and towers.

CHAPTER 6

STRUCTURAL TESTS AND INSPECTIONS

6.1 Special Inspections

6.1.1 General

The owner or the engineer or the architect incharge acting as the owner's agent shall employ one or more special inspectors who shall provide inspections during construction on the types of work listed under Section 6.1.5.

Exception: The building official may waive the requirement for the employment of a special inspector if the construction is of a minor nature.

6.1.2 Special Inspector

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection.

6.1.3 Duties and Responsibilities of the Special Inspector

The special inspector shall observe the work assigned for conformance to the approved design drawings and specifications.

The special inspector shall furnish inspection reports to the building official, the engineer or architect incharge and other designated persons. All discrepancies shall be brought to the immediate attention of the contractor for correction, then, if uncorrected, to the proper design authority and to the building official.

The special inspector shall submit a final signed report stating whether the work requiring special inspection was, to the best of the inspector's knowledge, in conformance to the approved plans and specifications and the applicable workmanship provisions of this code.

6.1.4 Standards of Quality

The standards adopted or recognized by Uniform Building Code or by any other internationally recognized building code shall be used for application of this chapter.

1. *Concrete:*

ASTM C 94, Ready-mixed Concrete

2. *Connections:*

Specification for Structural Joints Using ASTM A325 or A490 Bolts-Load and Resistance Factor Design, Research Council of Structural Connections, Section 6.1.5, Item 6

Specification for Structural Joints Using ASTM A325 or A490 Bolts-Allowable Stress Design, Research Council of Structural Connections, Section 6.1.5, Item 6

3. *Spray-applied Fire-resistive Materials:*

UBC Standard 7-6, Thickness and Density Determination for Spray-applied Fire-resistive Materials.

6.1.5 Types of Work

Except as provided in Section 6.1.1, the types of work listed below shall be inspected by a special inspector.

1. *Concrete:*

During taking of test specimens and placement of reinforced concrete, see Item 11 for shotcrete.

Exceptions:

1. Foundation concrete for Group R, Division 3 or Group U, Division 1 Occupancies, provided the building official finds that a special hazard does not exist (See Chapter 18, Table 18-I-C of UBC97).
2. For foundation concrete, other than cast-in-place drilled piles or caissons, where the structural design is based on an f_c' (as defined in Chapter 7) not greater than 17.2 MPa (2500 psi).
3. Nonstructural slabs on grade, including prestressed slabs on grade when effective prestress in concrete is less than 1.03 MPa (150 psi).
4. Site work concrete fully supported on earth and concrete where no special hazard exists.

2. *Bolts installed in concrete:*

Prior to and during the placement of concrete around bolts.

3. *Special moment-resisting concrete frame:*

For moment frames resisting design seismic load in structures within Seismic Zones 3 and 4, the special inspector shall provide reports to the person responsible for the structural design and shall provide continuous inspection of the placement of the reinforcement and concrete.

4. *Reinforcing steel and prestressing steel tendons:*

- 4.1 During all stressing and grouting of tendons in prestressed concrete.
- 4.2 During placement of reinforcing steel and prestressing tendons for all concrete required to have special inspection by Item 1.

Exception: The special inspector need not be present continuously during placing of reinforcing steel and prestressing tendons, provided the special inspector has inspected for conformance to the approved plans prior to the closing of forms or the delivery of concrete to the jobsite.

5. *Structural welding:*

5.1 General:

During the welding of any member or connection that is designed to resist loads and forces required by this code.

Exceptions:

1. Welding done in an approved fabricator's shop in accordance with Section 6.1.7.
2. The special inspector need not be continuously present during welding of the following items, provided the materials, qualifications of welding procedures and welders are verified prior to the start of work; periodic inspections are made of work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding:
 - 2.1 Single-pass fillet welds not exceeding 10 mm (5/16 inch) in size.
 - 2.2 Floor and roof deck welding.
 - 2.3 Welded studs when used for structural diaphragm or composite systems.
 - 2.4 Welded sheet steel for cold-formed steel framing members such as studs and joists.
 - 2.5 Welding of stairs and railing systems.

5.2 Special moment-resisting steel frames:

During the welding of special moment-resisting steel frames. In addition to Item 5.1 requirements, nondestructive testing as required by Section 6.3 of this chapter.

5.3 Welding of reinforcing steel:

During the welding of reinforcing steel.

Exception: The special inspector need not be continuously present during the welding of ASTM A706 reinforcing steel not larger than No. 5 bars used for embedments, provided the materials, qualifications of welding procedures and welders are verified prior to the start of work; periodic inspections are made of work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.

6. *High-strength bolting:*

The inspection of ASTM high-strength A325 and A490 bolts shall be in accordance with approved nationally recognized standards and the requirements of this section.

While the work is in progress, the special inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts; installation and tightening in such standards are met. Such inspections may be performed on a periodic basis in accordance with the requirements of Section 6.1.6. The special inspector shall observe the calibration procedures when such procedures are required by the plans or specifications and shall monitor the installation of bolts to determine that all plies of connected materials have been drawn together and that the selected procedure is properly used to tighten all bolts.

7. *Structural masonry:*

- 7.1 For masonry, other than fully grouted open-end hollow unit masonry, during preparation and taking of any required prisms or test specimens, placing of all masonry units, placement of reinforcement, inspection of grout space, immediately prior to closing of cleanouts, and during all grouting operations.

Exception: For hollow-unit masonry where the f'_{m} (as defined in Chapter 9) is no more than 10.5 MPa (1,500 psi) for concrete units or 18 MPa (2,610 psi) for clay units, special inspection may be performed as required for fully grouted open-end hollow-unit masonry specified in Item 7.2.

- 7.2 For fully grouted open-end hollow-unit masonry during preparation and taking of any required prisms or test specimens, at the start of laying units, after the placement of reinforcing steel, grout space prior to each grouting operation, and during all grouting operations.

8. *Insulating concrete fill:*

During the application of insulating concrete fill when used as part of a structural system.

Exception: The special inspections may be limited to an initial inspection to check the deck surface and placement of reinforcement. The special inspector shall supervise the preparation of compression test specimens during this initial inspection.

9. *Spray-applied fire-resistive materials:*

As required by UBC 1997 Standard 7-6.

10. *Piling, drilled piers and caissons:*

During driving and testing of piles and construction of cast-in-place drilled piles or caissons. See Items 1 and 4 for concrete and reinforcing steel inspection.

11. *Shotcrete:*

During taking of test specimens and placing of all shotcrete and as required by Sections 1924.10 and 1924.11 of UBC 1997.

Exception: Shotcrete work fully supported on earth, minor repairs and when, in the opinion of the building official, no special hazard exists.

12. *Special grading, excavation and filling:*

During earth-work excavations, grading and filling operations inspection to satisfy requirements of Chapter 18 and Appendix Chapter 33 of UBC 1997.

13. *Smoke-control system:*

- 13.1 During erection of ductwork and prior to concealment for the purposes of leakage testing and recording of device location.
13.2 Prior to occupancy and after sufficient completion for the purposes of pressure difference testing, flow measurements, and detection and control verification.

14. *Special cases:*

Work that, in the opinion of the building official, involves unusual hazards or conditions.

6.1.6 *Continuous and Periodic Special Inspection*

6.1.6.1 *Continuous special inspection*

Continuous special inspection means that the special inspector is on the site at all times observing the work requiring special inspection.

6.1.6.2 *Periodic special inspection*

Some inspections may be made on a periodic basis and satisfy the requirements of continuous inspection, provided this periodic scheduled inspection is performed as outlined in the project plans and specifications and approved by the building official.

6.1.7 *Approved Fabricators*

Special inspections required by this section and elsewhere in this code are not required where the work is done on the premises of a fabricator registered and approved by the building official to perform such work without special inspection. The certificate of registration shall be subject to revocation by the building official if it is found that any work done pursuant to the approval is in violation of this code. The approved fabricator shall submit a certificate of compliance that the work was performed in accordance with the approved plans and specifications to the building official and to the engineer or architect of record. The approved fabricator's qualifications shall be contingent on compliance with the following:

1. The fabricator has developed and submitted a detailed fabrication procedural manual reflecting key quality control procedures that shall provide a basis for inspection control of workmanship and the fabricator plant.
2. Verification of the fabricator's quality control capabilities, plant and personnel as outlined in the fabrication procedural manual, shall be by an approved inspection or quality control agency.
3. Periodic plant inspections shall be conducted by an approved inspection or quality control agency to monitor the effectiveness of the quality control program.
4. It shall be the responsibility of the inspection or quality control agency to notify the approving authority in writing of any change in the procedural manual. Any fabricator approval may be revoked for just cause. Reapproval of the fabricator shall be contingent on compliance with quality control procedures during the past year.

6.2 *Structural Observation*

Structural observation means the visual observation of the structural system for general conformance to the approved plans and specifications at significant construction stages and at completion of the structural system.

It shall be provided in Seismic Zone 2B, 3 or 4 when one of the following conditions exists:

1. The structure is defined in Chapter 5, Table 5.10 as Occupancy Categories 1, 2 or 3.
2. The structure belongs to Group B Office buildings and Group R Division 1 occupancies more than 25 m (82 ft) above the ground level. (See Chapter 5, Table 5.21).
3. The structure is in Seismic Zone 4, N_a as set forth in Chapter 5, Table 5.18 is greater than one and a lateral design is required for the entire structure.

Exception: One- and two-storey Group R, Division 3 and Group U Occupancies and one- and two-storey Groups B, F, M and S Occupancies.

4. When so designated by the architect or engineer of record, or
5. When such observation is specifically required by the building official.

The owner shall employ an engineer or architect responsible for the structural design to perform structural observation.

Observed deficiencies shall be reported in writing to the owner's representative, special inspector, contractor and the building official. The structural observer shall submit to the building official a written statement that the site visits have been made, identifying any reported deficiencies that, to the best of the structural observer's knowledge, have not been resolved.

6.3 Nondestructive Testing

In Seismic Zones 2B, 3 and 4, welded, fully restrained connections between the primary members of ordinary moment frames and special moment-resisting frames shall be tested by nondestructive methods for compliance with approved standards and job specifications. This testing shall be a part of the special inspection requirements of Section 6.1.5. A program for this testing shall be established by the person responsible for structural design and as shown on plans and specifications.

As a minimum, this program shall include the following:

1. All complete penetration groove welds contained in joints and splices shall be tested either by ultrasonic testing or by radiography.

Exceptions:

- a) When approved, the nondestructive testing rate for an individual welder or welding operator may be reduced but not below 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 915 mm (36 inch) in length where the effective throat thickness is 25 mm (1 inch) or less, each 305 mm (12 inch) increment or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 915 mm (36 inch) in length where the effective throat thickness is greater than 25 mm (1 inch), each 150 mm (6 inches) of length or fraction thereof shall be considered as one weld.
 - b) For complete penetration groove welds on materials less than 8 mm (5/16 inch) thick, nondestructive testing is not required; for this welding, continuous inspection is required.
 - c) When approved by the building official and outlined in the project plans and specifications, this nondestructive ultrasonic testing may be performed in the shop of an approved fabricator utilizing qualified test techniques in the employment of the fabricator.
2. Partial penetration groove welds when used in column splices shall be tested either by ultrasonic testing or radiography when required by the plans and specifications. For partial penetration groove welds when used in column splices, with an effective throat less than

20 mm (0.75 inch) thick, nondestructive testing is not required; for this welding, continuous special inspection is required.

3. Base metal thicker than 40 mm (1.5 inches), when subjected to through-thickness weld shrinkage strains, shall be ultrasonically inspected for discontinuities directly behind such welds after joint completion.

Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of approved national standards.

6.4 Prefabricated Construction

6.4.1 General

6.4.1.1 Purpose

The purpose of this section is to regulate materials and establish methods of safe construction where any structure or portion thereof is wholly or partially prefabricated.

6.4.1.2 Scope

Unless otherwise specifically stated in this section, all prefabricated construction and all materials used therein shall conform to all the requirements of this code.

6.4.1.3 Definition

Prefabricated Assembly is a structural unit, the integral parts of which have been built up or assembled prior to incorporation in the building.

6.4.2 Tests of Materials

Every approval of a material not specifically mentioned in this code shall incorporate as a proviso the kind and number of tests to be made during prefabrication.

6.4.3 Tests of Assemblies

The building official may require special tests to be made on assemblies to determine their durability and weather resistance.

6.4.4 Connections

See Section 5.11.11.1 for design requirements of connections for prefabricated assemblies.

6.4.5 Pipes and Conduits

See Section 5.11.11.2 for design requirements for removal of material for pipes, conduit and other equipment.

6.4.6 *Certificate and Inspection*

6.4.6.1 *Materials*

Materials and the assembly thereof shall be inspected to determine compliance with ASTM / UBC 1997 Standards. Every material shall be graded, marked or labeled where required as per ASTM / UBC 1997 Standards.

6.4.6.2 *Certificate*

A certificate of approval shall be furnished with every prefabricated assembly, except where the assembly is readily accessible to inspection at the site. The certificate of approval shall certify that the assembly in question has been inspected and meets all the requirements of this code. When mechanical equipment is installed so that it cannot be inspected at the site, the certificate of approval shall certify that such equipment complies with the laws applying thereto.

6.4.6.3 *Certifying agency*

To be acceptable under this code, every certificate of approval shall be made by an approved agency.

6.4.6.4 *Field erection*

Placement of prefabricated assemblies at the building site shall be inspected by the building official to determine compliance with this code.

6.4.6.5 *Continuous inspection*

If continuous inspection is required for certain materials where construction takes place on the site, it shall also be required where the same materials are used in prefabricated construction.

Exception: Continuous inspection will not be required during prefabrication if the approved agency certifies the construction and furnishes evidence of compliance.

CHAPTER 7**STRUCTURAL CONCRETE****7.1 Symbols and Notations**

A_{ch}	=	Cross-sectional area of a structural member measured out-to-out of transverse reinforcement, mm ² (in ²).
A_{cv}	=	Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm ² (in ²).
A_{cw}	=	Area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, mm ² (in ²).
A_{vd}	=	Total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm ² (in ²).
A_g	=	Gross area of concrete section. For hollow section, A_g is the area of concrete and does not include the area of the void(s), mm ² (in ²).
$A_{s, min}$	=	Minimum area of flexural reinforcement, mm ² (in ²).
A_s	=	Longitudinal reinforcement, mm ² (in ²).
A_{st}	=	Total area of nonprestressed longitudinal reinforcement (bars or steel shapes), mm ² (in ²).
A_{sh}	=	Total cross-sectional area of transverse reinforcement (including cross-ties) within spacing 's' and perpendicular to dimension ' b_c ', mm ² (in ²).
A_j	=	Effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, mm ² (in ²).
b	=	Width of beam or column, mm (in).
b_c	=	Cross-sectional dimension of column core measured centre of outer leg of the transverse reinforcement comprising area " A_{sh} ", mm (in).
b_w	=	Width of member or diameter of circular section, mm (in).
b_o	=	Perimeter of the critical section for shear in slabs and footings, mm (in).
c	=	Distance from extreme compression fiber to neutral axis, mm (in).
c_1	=	Dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm (in).
c_t	=	Distance from the interior face of the column to the slab edge measured parallel to c_1 , but not exceeding c_1 , mm (in).
D	=	Dead loads, or related internal moments and forces
d	=	Effective depth of beam/column, mm (in).
d'	=	Distance from the extreme compression fiber to centroid of the longitudinal compression reinforcement, mm (in).
d_b	=	Nominal diameter of bar or wire, mm (in).
d_s	=	Distance of extreme tension fiber to centroid of tension reinforcement, mm (in).
E	=	Load effects of earthquake, or related internal moments and forces
f'_c	=	Concrete compressive cylinder strength at 28 days, MPa (psi).
f_y	=	Specified yield strength of reinforcement, MPa (psi).
f_{yt}	=	Specified yield strength f_y of transverse reinforcement, MPa (psi).
h	=	Overall height or thickness of member, mm (in).
h_x	=	Maximum centre-to-centre horizontal spacing of cross-ties or hoop legs on all faces of the column, mm (in).
h_w	=	Height of entire wall from base to top or height of the segment of wall considered, mm (in).
L	=	Live loads, or related internal moments and forces.
l_d	=	Development length in tension of deformed bar, deformed wire, plain and

l_{dh}	=	deformed welded wire reinforcement, mm (in).
	=	Development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), mm (in).
l_n	=	Length of clear span measured face-to-face of supports, mm (in).
l_o	=	Length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm (in).
l_w	=	Length of entire wall or length of segment of wall considered in direction of shear force, mm (in).
M_n	=	Nominal flexural strength at section, N-mm (in-lb).
M_u	=	Factored moment at section, N-mm (in-lb).
M_{pr}	=	Probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor, ϕ , of 1.0, N-mm (in-lb).
M_{nb}	=	Nominal flexural strength of beam including slab where in tension, framing into joint, N-mm (in-lb).
M_{nc}	=	Nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength, N-mm (in-lb).
M_{slab}	=	Portion of slab factored moment balanced by support moment, N-mm (in-lb).
P_b	=	Nominal axial strength at balanced strain condition, N (lb).
P_n	=	Nominal axial strength of cross-section, N (lb).
P_o	=	Nominal axial strength at zero eccentricity, N (lb).
P_u	=	Factored axial force; to be taken as positive for compression and negative for tension, N (lb).
S	=	Snow load, or related internal moments and forces.
S_e	=	Moment, shear, or axial force at connection corresponding to development of probable strength at intended yield locations, based on the governing mechanism of inelastic lateral deformation, considering both gravity and earthquake load effects.
S_n	=	Nominal flexural, shear, or axial strength of connection.
S_y	=	Yield strength of connection, based on f_y , for moment, shear, or axial force.
s	=	Center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement.
s_o	=	Center-to-center, spacing of transverse reinforcement within the length l_o .
V_c	=	Nominal shear strength provided by concrete, N (lb).
V_e	=	Design shear force corresponding to the development of the probable moment strength of the member.
V_u	=	Factored shear force at section, N (lb).
V_n	=	Nominal shear strength, N (lb).
v_n	=	Nominal shear stress, MPa (psi).
α	=	Angle defining the orientation of reinforcement.
α_c	=	Coefficient defining the relative contribution of concrete strength to nominal wall shear strength.
ρ	=	Ratio of A_s to bd .
ρ_l	=	Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement.
ρ_t	=	Ratio of area distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement.

ρ_s	=	Ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals).
ϕ	=	Strength reduction factors.
γ_f	=	Factor used to determine the unbalanced moment transferred by flexure at slab-column connections.
δ_u	=	Design displacement, mm (in).

7.2 Definitions

Base of structure. Level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

Boundary elements. Portions along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements as required by Section 7.8.6 or 7.10.5.3.

Collector elements. Elements that serve to transmit the inertial forces within structural diaphragms to members of the lateral-force-resisting systems.

Connection. A region that joins two or more members, of which one or more is precast.

Ductile connection. Connection that experiences yielding as a result of the design displacements.

Strong connection. Connection that remains elastic while adjoining members experience yielding as a result of the design displacements.

Crosstie. A continuous reinforcing bar having a seismic hook at one end and a hook not less than 90 degrees with at least a six-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90 degree hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

Design displacement. Total lateral displacement expected for the design-basis earthquake, as required by the governing code for earthquake-resistant design.

Design load combinations. Combinations of factored loads and forces in Chapter 5.

Design storey drift ratio. Relative difference of design displacement between the top and bottom of a storey, divided by the storey height.

Development length for a bar with a standard hook. The shortest distance from the critical section (where the strength of the bar is to be developed) to the outside end of the 90 degree hook.

Factored loads and forces. Loads and forces multiplied by appropriate load factors in Chapter 5.

Hoop. A closed tie or continuously wound tie. A closed tie can be made up of several reinforcement elements each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

Joint. Portion of structure common to intersecting members. The effective cross-sectional area of the joint, A_j , for shear strength computations is defined in Section 7.6.3.1.

Lateral-force resisting system. That portion of the structure composed of members proportioned to resist forces related to earthquake effects.

Lightweight aggregate concrete. All-lightweight or sand-lightweight aggregate concrete made with lightweight aggregates conforming to Section 3.3 of ACI 318 – 05.

Moment frame. Frame in which members and joints resist forces through flexure, shear, and axial force. Moment frames shall be categorized as follows:

Intermediate moment frame. A cast-in-place frame complying with the requirements of 7.3.2.3 and 7.13 in addition to the requirements for ordinary moment frames.

Ordinary moment frame. A cast-in-place or precast concrete frame complying with the requirements of Chapters 1 through 18 of ACI 318 – 2005.

Special moment frame. A cast-in-place frame complying with the requirements of 7.3.2.3, 7.3.3 through 7.3.7, and 7.4 through 7.6 or a precast frame complying with the requirements of 7.3.2.3, 7.3.3 through 7.3.7, and 7.4 through 7.7. In addition, the requirements for ordinary moment frames shall be satisfied.

Plastic hinge region. Length of frame element over which flexural yielding is intended to occur due to design displacements, extending not less than a distance h from the critical section where flexural yielding initiates.

Seismic hook. A hook on a stirrup, hoop, or crosstie having a bend not less than 135 degrees, except that circular hoops shall have a bend not less than 90 degrees. Hooks shall have a six-diameter [but not less than 75 mm (3 in.)] extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

Special boundary elements. Boundary elements required by 7.8.6.2 or 7.8.6.3.

Specified lateral forces. Lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake-resistant design.

Structural diaphragms. Structural members, such as floor and roof slabs, that transmit inertial forces to lateral-force resisting members.

Structural trusses. Assemblages of reinforced concrete members subjected primarily to axial forces.

Structural walls. Walls proportioned to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shearwall is a structural wall. Structural walls shall be categorized as follows:

Intermediate precast structural wall. A wall complying with all applicable requirements of Chapters 1 through 18 of ACI 318 – 05 in addition to 7.14.

Ordinary reinforced concrete structural wall. A wall complying with the requirements of Chapters 1 through 18 of ACI 318 – 05.

Special precast structural wall. A precast wall complying with the requirements of 7.9. In addition, the requirements for ordinary reinforced concrete structural walls and the requirements of 7.3.2.3, 7.3.3 through 7.3.7, and 7.8 shall be satisfied.

Special reinforced concrete structural wall. A cast-in-place wall complying with the requirements of 7.3.2.3, 7.3.3 through 7.3.7, and 7.8 in addition to the requirements for ordinary reinforced concrete structural walls.

Strut. An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

Tie elements. Elements that serve to transmit inertia forces and prevent separation of building components such as footings and walls.

7.3 General Requirements

7.3.1 Scope

7.3.1.1 This section contains special requirements for design and construction of cast-in-place reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response as specified in Chapter 5. For applicable specified concrete compressive strengths see Section 1.1.1 of ACI 318-05 and Section 7.3.4.1. For explanation of provisions, see Chapter 21, Commentary of ACI 318-05. All equations are in SI units whereas equations given in parenthesis are in FPS units.

7.3.1.2 In regions of low seismic risk (Seismic Zone 1) or for structures assigned to low seismic performance or design categories, the provisions of Chapters 1 through 18 and 22 of ACI 318-05 shall apply. Where the design seismic loads are computed using provisions for intermediate or special concrete systems, the requirements of Chapter 7 for intermediate, or special system shall be satisfied.

7.3.1.3 In regions of moderate seismic risk (Seismic Zones 2A, 2B) or for structures assigned to intermediate seismic performance or design categories, intermediate or special moment frames, or ordinary, intermediate, or special structural walls, shall be used to resist forces induced by earthquake motions. Where the design seismic loads are computed using provisions for special concrete system, the requirements of Chapter 7 for special system shall be satisfied.

7.3.1.4 In regions of high seismic risk (Seismic Zones 3, 4) or for structures assigned to high seismic performance or design categories, special moment frames, special structural walls, and diaphragms and trusses complying with 7.3.2 through 7.3.6 and 7.4 through 7.11 shall be used to resist forces induced by earthquake motions. Member not proportioned to resist earthquake forces shall comply with 7.12.

7.3.1.5 A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter.

7.3.2 Analysis and Proportioning of Structural Members

7.3.2.1 The interaction of all structural and nonstructural members that materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

7.3.2.2 Rigid members assumed not to be a part of the lateral-force resisting system shall be permitted provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members, which are not a part of the lateral force resisting system, shall also be considered.

7.3.2.3 Structural members below base of structure that are required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of Chapter 7.

7.3.2.4 All structural members assumed not to be part of lateral-force resisting system shall conform to 7.12.

7.3.3 *Strength Reduction Factors*

7.3.3.1 Design strength provided by a member, its connection to other members and its cross sections, in terms of flexure, axial load, shear and tension, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by a strength reduction factor ϕ in Section 7.3.3.2 a, b, c, d, and f.

7.3.3.2 Strength-reduction factor ϕ shall be as follows:

a)	Flexure, without axial load.....	0.90
b)	Axial load and axial load with flexure. (For axial load with flexure, both axial load and moment nominal strength shall be multiplied by appropriate single value of ϕ .)	
	Axial tension and axial tension with flexure	0.90
	Axial compression and axial compression with flexure:	
	Members with spiral reinforcement	0.75
	Other reinforced members	0.70

Except that for low values of axial compression, ϕ shall be permitted to be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi (413.7MPa), with symmetric reinforcement, and with $(h-d'-d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ to zero.

For other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10 f_c' A_g$ or ϕP_b , whichever is smaller, to zero.

c) Shear and torsion (See also "e" for shear walls and frames in seismic zones 3 and 4) 0.85

d) Bearing on concrete 0.70

e) In seismic zones 3 and 4, strength reduction factors ϕ shall be as given above except for the following:

i) The strength reduction factor shall be 0.6 for the design of walls. Topping slabs used as diaphragms over precast concrete members and structural framing members, with the exception of joints, if their nominal strength is less than the shear corresponding to development of their nominal flexural strength. The nominal flexural strength shall be determined corresponding to the most critical factored axial loads including earthquake effects. The shear reduction factor for joints shall be 0.85.

ii) Reinforcement used for diaphragms chords or collectors placed in topping slabs over precast concrete members shall be designed using a strength reduction factor of 0.6.

f) Strength reduction factor for flexure compression, shear and bearing of structural plain concrete shall be 0.65.

7.3.4 *Concrete in Members Resisting Earthquake Induced Forces*

7.3.4.1 Specified compressive cylinder strength of concrete f'_c at 28 days, shall be not less than 21 MPa (3,000 psi)

7.3.4.2 Specified compressive cylinder strength of lightweight concrete, f'_c at 28 days, shall not exceed 35 MPa (5,000 psi) unless demonstrated by experimental evidence that structural members made with that light weight concrete provide strength and toughness equal to or exceeding those of comparable members made with normal weight concrete of the same strength.

7.3.5 *Reinforcement in Members Resisting Earthquake Induced Forces*

Reinforcement resisting earthquake-induced flexural and axial forces in frame members and in structural wall boundary elements lying in seismic zones 3 and 4 shall comply with ASTM A706. ASTM A615, Grades 40 and 60 reinforcement, shall be permitted in these members if:

- (a) The actual yield strength based on mill tests does not exceed f_y by more than 124 MPa (18,000 psi) retests shall not exceed this value by more than an additional 21 MPa (3,000 psi); and
- (b) The ratio of the actual tensile strength to the actual yield strength is not less than 1.25.

The value of f_{yt} for transverse reinforcement including spiral reinforcement shall not exceed 420 MPa (60,000 psi).

7.3.6 *Welded Splices*

7.3.6.1 Welded splices in reinforcement resisting earthquake-induced forces shall conform to 12.14.3.4 of ACI 318-05 and shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements.

7.3.6.2 Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design shall not be permitted.

7.3.7 *Anchoring to Concrete*

Anchors resisting earthquake-induced forces in structures in regions of moderate to high seismic risk shall conform to the additional requirements D.3.3 of Appendix D of ACI 318-05.

7.4 *Flexural Members of Special Moment Frames*

7.4.1 *Scope*

Requirements of this section shall apply to special moment frame members (a) resisting earthquake-induced forces and (b) proportioned primarily to resist flexure. These frame members shall also satisfy following conditions:

7.4.1.1 Factored axial compressive force on the member, P_u , shall not exceed $A_g f'_c / 10$.

7.4.1.2 Clear span for member, l_n , shall not be less than four times its effective depth.

7.4.1.3 Width of member, b_w , shall not be less than the smaller of $0.3h$ and 250 mm (10 in).

7.4.1.4 Width of member, b_w , shall not exceed width of supporting member (measured on a plane perpendicular to the longitudinal axis of flexural member) plus distances on each side of supporting member not exceeding three-fourths of the depth of flexural member.

7.4.2 *Longitudinal Reinforcement*

7.4.2.1 At any section of a flexural member, except as provided in 10.5.3 of ACI 318-05, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10-3) of ACI 318-05 but not less than $200b_wd/f_y$, and the reinforcement ratio, ρ , shall not exceed 0.025. At least two bars shall be provided continuously both top and bottom.

7.4.2.2 Positive moment strength at joint face shall be not less than one-half of the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along member length shall be less than one-fourth the maximum moment strength provided at face of either joint.

7.4.2.3 Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Spacing of the transverse reinforcement enclosing the lapped bars shall not exceed the smaller of $d/4$ and 100 mm (4 in). Lap splices shall not be used:

- (a) Within the joints;
- (b) Within a distance of twice the member depth from the face of the joint; and
- (c) Where analysis indicates flexural yielding is caused by inelastic lateral displacements of the frame.

7.4.2.4 Welded splices shall conform to 7.3.6.

7.4.3 *Transverse Reinforcement*

7.4.3.1 Hoops shall be provided in the following regions of frame members:

- a) Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural member;
- b) Over lengths equal to twice the member depth on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.

7.4.3.2 The first hoop shall be located not more than 50 mm (2 in.) from the face of a supporting member.

Spacing of the hoops shall not exceed the smallest of a, b, c and d:

- a) $d/4$;
- b) eight times the diameter of the smallest longitudinal bars;
- c) 24 times the diameter of the hoop bars; and
- d) 300 mm (12 in).

7.4.3.3 Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 7.10.5.3 of ACI 318-05.

7.4.3.4 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$ throughout the length of the member.

7.4.3.5 Stirrups or ties required to resist shear shall be hoops over length of members in 7.4.3, 7.5.4 and 7.6.2.

7.4.3.6 Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90 degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90 degree hooks of the crossties shall be placed on that side.

7.4.4 *Shear Strength Requirements*

7.4.4.1 *Design forces*

The design shear force, V_e , shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable flexural moment strength, M_{pr} , act at the joint faces and that the member is loaded with the factored tributary gravity load along its span.

7.4.4.2 *Transverse reinforcement*

Transverse reinforcement over the lengths identified in 7.4.3.1 shall be proportioned to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

- (a) The earthquake-induced shear force calculated in accordance with 7.4.4.1 represents one-half or more of the maximum required shear strength within those lengths;
- (b) The factored axial compressive force, P_u , including earthquake effects is less than $A_g f'_c / 20$.

7.5 Special Moment Frame Members Subjected to Bending and Axial Load

7.5.1 *Scope*

The requirements of this sub-section apply to special moment frame members (a) resisting earthquake induced forces and (b) having a factored axial compressive force P_u exceeding $A_g f'_c / 10$. These members shall also satisfy the conditions of 7.5.1.1 and 7.5.1.2.

7.5.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 300 mm (12 in).

7.5.1.2 The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

7.5.2 *Minimum Flexural Strength of Columns*

7.5.2.1 Flexural strength of any column proportioned to resist P_u exceeding $A_g f'_c / 10$ shall satisfy 7.5.2.2 or 7.5.2.3.

Lateral strength and stiffness of columns not satisfying 7.5.2.2 shall be ignored when determining calculated strength and stiffness of the structure but such columns shall conform to 7.12.

7.5.2.2 The flexural strengths of the columns shall satisfy:

$$\sum M_{nc} \geq \left(\frac{6}{5}\right) \sum M_{nb} \quad (7.5-1)$$

$\sum M_{nc}$ = sum of nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_{nb}$ = sum of nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab is in tension under moments at the face of the joint, slab reinforcement within an effective slab width as defined in ACI 318-05, section 8.10 and shall be assumed to contribute to M_{nb} if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation 7.5-1 shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

7.5.2.3 If equation 7.5-1 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in 7.5.4.1 through 7.5.4.3 over their full height.

7.5.3 *Longitudinal Reinforcement*

7.5.3.1 Area of longitudinal reinforcement, A_{st} , shall not be less than $0.01A_g$ or more than $0.06A_g$.

7.5.3.2 Welded splices shall conform to 7.3.6. Lap splices shall be only permitted within center half of the member length, shall be designed as tension lap splices and shall be enclosed within transverse reinforcement conforming to 7.5.4.2 and 7.5.4.3.

7.5.4 *Transverse Reinforcement*

7.5.4.1 Transverse reinforcement required in (a) through (e) shall be provided unless a larger amount is required by 7.5.3.2 or 7.5.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s , shall not be less than required by

$$\rho_s = 0.12 \frac{f'_c}{f'_{yt}} \quad (7.5-2)$$

and shall not be less than

$$\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f'_{yt}} \quad (7.5-3)$$

(b) The total cross-sectional area of rectangular hoop reinforcement, A_{sh} , shall not be less than required by equations (7.5-4) and (7.5-5).

$$A_{sh} = 0.3 \left(\frac{sb_c f'_c}{f_{yt}} \right) \left[\left(\frac{A_g}{A_{ch}} \right) - 1 \right] \quad (7.5-4)$$

$$A_{sh} = 0.09s \left(\frac{b_c f'_c}{f_{yt}} \right) \quad (7.5-5)$$

- (c) Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement.
- (d) If the design strength of member core satisfies the requirement of the design loading combinations including earthquake effect, equations (7.5-3) and (7.5-4) need not be satisfied.
- (e) If the thickness of the concrete outside the confining transverse reinforcement exceeds 100 mm (4 in.), additional transverse reinforcement shall be provided at a spacing not exceeding 300 mm (12 in). Concrete cover on the additional reinforcement shall not exceed 100 mm (4 in).

7.5.4.2 Spacing of transverse reinforcement shall not exceed the smallest of (a), (b), and (c):

- (a) one-quarter of the minimum member dimension;
- (b) six times the diameter of the longitudinal reinforcement; and
- (c) s_o , as defined by Eq. (7.5-6)

$$s_o = 4 + \left(\frac{14 - h_x}{3} \right) \quad (7.5-6)$$

The value of s_o shall not exceed 150 mm (6 in.) and need not be taken less than 100 mm (4 in.).

7.5.4.3 Horizontal spacing of cross ties or legs of overlapping hoops, h_x , shall not exceed 350 mm (14 in.) on center.

7.5.4.4 Transverse reinforcement as specified in 7.5.4.1 through 7.5.4.3 shall be provided over a length l_o from each joint face and on both sides of any section where flexural yielding is likely to occur as a result of inelastic lateral displacements of the frame. Length l_o shall not be less than the largest of (a), (b), and (c):

- (a) The depth of the member at the joint face or at the section where flexural yielding is likely to occur;
- (b) one-sixth of the clear span of the member; and
- (c) 450 mm (18 in.).

7.5.4.5 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as required in 7.5.4.1 through 7.5.4.3 over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, related to earthquake effect, exceeds $A_g f'_c / 10$. Transverse reinforcement as required in 7.5.4.1 through 7.5.4.3 shall extend at least the development length in tension, l_d , into discontinued member, where l_d is determined in accordance of 7.6.4 using the largest longitudinal reinforcement in the column. If the lower end of the column terminates on a wall, transverse reinforcement as required in 7.5.4.1 through 7.5.4.3 shall extend into wall at least l_d of the largest longitudinal column

bar at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as required in 7.5.4.1 through 7.5.4.3 shall extend at least 300 mm (12 in) into the footing or mat.

7.5.4.6 Where transverse reinforcement, as specified in 7.5.4.1 through 7.5.4.3, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with center-to-center spacing ‘*s*’ not exceeding the smaller of six times the diameter of the longitudinal column bars and 150 mm (6 in).

7.5.5 *Shear Strength Requirements*

7.5.5.1 *Design forces*

The design shear force, V_e , shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths, M_{pr} , at each end of the member associated with the range of factored axial loads, P_u , acting on the member. The member shears need not exceed those determined from joint strengths based on M_{pr} of the transverse members framing into the joint. In no case shall V_e be less than the factored shear determined by analysis of the structure.

7.5.5.2 Transverse reinforcement over the lengths l_o , identified in 7.5.5.1, shall be proportioned to resist shear assuming $V_c = 0$ when both (a) and (b) occur:

- (a) The earthquake-induced shear force represents one-half or more of the maximum required shear strength within l_o ;
- (b) The factored axial compressive force, P_u , including earthquake effects is less than $A_g f'_c / 20$.

7.6 *Joints of Special Moment Frames*

7.6.1 *General Requirements*

7.6.1.1 Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25 f_y$.

7.6.1.2 Strength of joint shall be governed by the appropriate ϕ factors given in 7.3.3.2 (e).

7.6.1.3 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 7.6.4 and in compression according to Chapter 12, ACI 318- 2005.

7.6.1.4 Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar for normal weight concrete. For lightweight concrete, the dimension shall be not less than 26 times the bar diameter.

7.6.2 *Transverse Reinforcement*

7.6.2.1 Transverse hoop reinforcement in 7.5.4 shall be provided within the joint, unless the joint is confined by structural members in 7.7.2.2.

7.6.2.2 Within h of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by 7.4.4.1 shall be provided where members frame into all four sides

of the joint and where each member width is at least three-fourth the column width. At these locations, the spacing required in 7.5.4.2 shall be permitted to be increased to 150 mm (6 in.).

7.6.2.3 Transverse reinforcement as required by 7.5.4 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

7.6.3 Shear Strength

7.6.3.1 Nominal shear strength V_n of the joint shall not be taken as greater than the values specified below for normal weight concrete.

$$\text{For joints confined on all four faces } 1.7 \sqrt{f'_c} A_j \text{ (SI Units)} \quad (20 \sqrt{f'_c} A_j) \text{ (FPS Units)}$$

$$\text{For joints confined on three faces or on two opposite faces } 1.2 \sqrt{f'_c} A_j \text{ (SI Units)} \quad (15 \sqrt{f'_c} A_j) \text{ (FPS Units)}$$

$$\text{For others } 1.0 \sqrt{f'_c} A_j \text{ (SI Units)} \quad (12 \sqrt{f'_c} A_j) \text{ (FPS Units)}$$

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

A_j is the effective cross-sectional area within a joint computed from joint depth times effective joint width. Joint depth shall be the overall depth of the column. Effective joint width shall be the overall width of the column, except where a beam frames into a wider column, effective joint width shall not exceed the smaller of (a) and (b):

- (a) Beam width plus joint depth
- (b) Twice the smaller perpendicular distance from longitudinal axis of beam to column side.

7.6.3.2 For lightweight aggregate concrete, the nominal shear strength of the joint shall not exceed three-quarters of the limits given in 7.6.3.1.

7.6.4 Development Length of Bars in Tension

7.6.4.1 The development length, l_{dh} , for a bar with a standard 90 degree hook in normal weight concrete shall not be less than the largest of $8d_b$, 150 mm (6 in.), and the length required by Eq. (7.6-1).

$$l_{dh} = \frac{f_y d_b}{5.4 \sqrt{f'_c}} \text{ (SI Units)} \quad (l_{dh} = \frac{f_y d_b}{65 \sqrt{f'_c}}) \text{ (FPS Units)} \quad (7.6-1)$$

for bar sizes # 3 through #11.

For lightweight concrete, l_{dh} for a bar with a standard 90 degree hook shall not be less than the largest of $10 d_b$, 190 mm (7.5 in.), and 1.25 times the length required by Eq. (7.6-1).

The 90 degree hook shall be located within the confined core of a column or of a boundary element.

7.6.4.2 For bar sizes #3 through #11, l_d , the development length in tension for a straight bar, shall not be less than the larger of (a) and (b):

- (a) 2.5 times the length required by 7.6.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm (12 in.).
- (b) 3.25 times the length required by 7.6.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm (12 in.).

7.6.4.3 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of l_d not within the confined core shall be increased by a factor of 1.6.

7.6.4.4 If epoxy-coated reinforcement is used, the development lengths in 7.6.4.1 through 7.6.4.3 shall be multiplied by the applicable factor given in ACI 318-05, Chapter 12, Sec 12.2.4 or 12.5.2.

7.7 Special Moment Frames Constructed using Precast Concrete

7.7.1 Special moment frames with ductile connections constructed using precast concrete shall satisfy (a) and (b) and all requirements for special moment frames constructed with cast-in-place concrete:

- (a) V_n for connections computed according to 11.7.4 of ACI 318-05 shall not be less than $2V_e$, where V_e is calculated according to Sections 7.4.4.1 or 7.5.5.1;

7.7.2 Special moment frames with strong connections constructed using precast concrete shall satisfy all requirements for special moment frames constructed with cast-in-place concrete, as well as (a), (b), (c), and (d).

- (a) Provisions of 7.4.1.2 shall apply to segments between locations where flexural yielding is intended to occur due to design displacements;
- (b) Design strength of the strong connection, ϕS_n , shall not be less than S_e ;
- (c) Primary longitudinal reinforcement shall be made continuous across connections and shall be developed outside both the strong connection and the plastic hinge region; and
- (d) For column-to-column connections, ϕS_n shall not be less than $1.4S_e$. At column-to-column connections, ϕM_n shall be not less than $0.4M_{pr}$ for the column within the storey height, and ϕV_n of the connection shall be not less than V_e determined by 7.5.5.1.

7.7.3 Special moment frames constructed using precast concrete and not satisfying the requirements of 7.7.1 or 7.6.2 shall satisfy the requirements of ACI T1.1, "Acceptance Criteria for Moment Frames Based on Structural Testing," and the requirements of (a) and (b):

- (a) Details and materials used in the test specimens shall be representative of those used in the structure and
- (b) The design procedure used to proportion the test specimens shall define the mechanism by which the frame resists gravity and earthquake effects, and shall establish acceptance values for sustaining that mechanism. Portions of the mechanism that deviate from code requirements shall be contained in the test specimens and shall be tested to determine upper bounds for acceptance values.

7.8 Special Reinforced Concrete Structural Walls and Coupling Beams

7.8.1 Scope

The requirements of this section apply to special reinforced concrete structural walls and coupling beams serving as part of the earthquake force-resisting system.

7.8.2 Reinforcement

7.8.2.1 The distributed web reinforcement ratios, ρ_l and ρ_t , for structural walls shall not be less than 0.0025, except that if V_u does not exceed $A_{cv}\sqrt{f'_c}$, ρ_l and ρ_t shall be permitted to be reduced to the values required in ACI 318-05, Section 14.3. Reinforcement spacing each way in structural walls shall not exceed 450 mm (18 in.) Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

7.8.2.2 At least two curtains of reinforcement shall be used in a wall if V_u exceeds $0.17A_{cv}\sqrt{f'_c}$.

7.8.2.3 Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with ACI318-2005, Chapter 12, except:

- a) The effective depth of the member shall be permitted to be 0.8 l_w for walls.
- b) The requirements of 12.11, 12.12 and 12.13 of ACI 318 – 2005 need not be satisfied.
- c) At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for f_y in tension.

7.8.3 Design Forces

V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

7.8.4 Shear Strength

7.8.4.1 V_n of structural walls shall not exceed Eq.(7.8-1) where the coefficient α_c is 0.25 (3.0) for $h_w/l_w \leq 1.5$, is 0.17 (2.0) for $h_w/l_w \geq 2.0$ and varies linearly between 0.25 (3.0) and 0.17 (2.0) for h_w/l_w between 1.5 and 2.0.

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho f_y) \quad (7.8-1)$$

7.8.4.2 In 7.8.4.1, the value of ratio h_w / l_w used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

7.8.4.3 Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If h_w / l_w does not exceed 2.0, reinforcement ratio ρ_l shall not be less than reinforcement ratio ρ_t .

7.8.4.4 For all wall piers sharing a common lateral force, V_n shall not be taken larger than

$$0.66 A_{cv}\sqrt{f'_c} \quad (\text{SI Units}) \quad (8 A_{cv}\sqrt{f'_c}) , (\text{FPS Units})$$

where A_{cv} is the gross area of concrete bounded by web thickness and length of section. For any one of the individual wall piers, V_n shall not be taken larger than

$$0.83 A_{cw} \sqrt{f'_c} \quad (\text{SI Units}) \quad (10 A_{cw} \sqrt{f'_c}) , (\text{FPS Units})$$

where A_{cw} is the area of concrete section of the individual pier considered.

7.8.4.5 For horizontal wall segments and coupling beams, V_n shall not be taken larger than

$$0.83 A_{cw} \sqrt{f'_c} \quad (\text{SI Units}) \quad (10 A_{cw} \sqrt{f'_c}) , (\text{FPS Units})$$

where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam. Horizontal wall segment refers to a part of a wall bounded by openings or by an opening and an edge.

7.8.5 *Design for Flexure and Axial Loads*

7.8.5.1 Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with ACI 318-05 Sections 10.2 and 10.3 except that 10.3.6 and the nonlinear strain requirements of ACI 318-05 Section 10.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

7.8.5.2 Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

7.8.6 *Boundary Elements of Special Reinforced Concrete Structural Walls*

7.8.6.1 The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with 7.8.6.2 or 7.8.6.3. The requirements of 7.8.6.4 and 7.8.6.5 also shall be satisfied.

7.8.6.2 This section applies to walls or wall piers that are effectively continuous from the base of structure to top of wall and designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by 7.8.6.3.

(a) Compression zones shall be reinforced with special boundary elements where:

$$c \geq \frac{l_w}{600(\delta_u / h_w)} \quad (7.8-2)$$

c in Eq. (7.8-2) corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u . Ratio δ_u / h_w in Eq. 7.8-2 shall not be taken less than 0.007.

(b) Where special boundary elements are required by 7.8.6.2(a), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $M_u / 4V_u$.

7.8.6.3 Structural walls not designed to the provisions of 7.8.6.2 shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to factored forces including E , load effects for earthquake

effect, exceeds $0.2f'_c$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in 7.8.5.2 shall be used.

7.8.6.4 Where special boundary elements are required by 7.8.6.2 or 7.8.6.3, (a) through (e) shall be satisfied:

- (a) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1l_w$ and $c/2$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u ;
- (b) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm (12 in.) into the web;
- (c) Special boundary element transverse reinforcement shall satisfy the requirements of 7.4.4.1 through 7.5.4.3, except Eq. (7.5-4) need not be satisfied;
- (d) Special boundary element transverse reinforcement at the wall base shall extend into the support at least the development length of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 300 mm (12 in.) into the footing or mat;
- (e) Horizontal reinforcement in the wall web shall be anchored to develop f_y within the confined core of the boundary element;

7.8.6.5 Where special boundary elements are not required by 7.8.6.2 or 7.8.6.3, (a) and (b) shall be satisfied:

- (a) If the longitudinal reinforcement ratio at the wall boundary is greater than $2.8/f_y$ ($400/f_y$), boundary transverse reinforcement shall satisfy 7.5.4.1(c), 7.5.4.3, and 7.8.6.4(a). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 200 mm (8 in);
- (b) Except when V_u in the plane of the wall is less than $0.083A_{cv}\sqrt{f'_c}$ ($A_{cv}\sqrt{f'_c}$), horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

7.8.7 Coupling Beams

7.8.7.1 Coupling beams with aspect ratio, $(l_n / h) \geq 4$, shall satisfy the requirements of 7.4. The provisions of 7.4.1.3 and 7.4.1.4 need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

7.8.7.2 Coupling beams with aspect ratio, $(l_n / h) < 4$, shall be permitted to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

7.8.7.3 Coupling beams with aspect ratio, $(l_n / h) < 2$, and with V_u exceeding $0.33A_{cw}\sqrt{f'_c}$ ($4A_{cw}\sqrt{f'_c}$) shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying capacity of the structure, or the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

7.8.7.4 Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy (a) through (f):

- (a) Each group of diagonally placed bars shall consist of a minimum of four bars assembled in a core having sides measured to the outside of transverse reinforcement no smaller than $b_w / 2$ perpendicular to the plane of the beam and $b_w / 5$ in the plane of the beam and perpendicular to the diagonal bars;
- (b) V_n shall be determined by

$$V_n = 2A_{vd}f_y \sin\alpha \leq 0.83\sqrt{f'_c}A_{cw} \quad (V_n = 2A_{vd}f_y \sin\alpha \leq 10\sqrt{f'_c}A_{cw}) \quad (7.8-3)$$

where α is the angle between the diagonally placed bars and the longitudinal axis of the coupling beam.

- (c) Each group of diagonally placed bars shall be enclosed in transverse reinforcement satisfying 7.5.4.1 through 7.5.4.3. For the purpose of computing A_g for use in Eq. (7.5-3) and Eq. (7.5-5), the minimum concrete cover as required in 7.7 of ACI 318-05 shall be assumed on all four sides of each group of diagonally placed reinforcing bars;
- (d) The diagonally placed bars shall be developed for tension in the wall;
- (e) The diagonally placed bars shall be considered to contribute to M_n of the coupling beam;
- (f) Reinforcement parallel and transverse to the longitudinal axis shall be provided and, as a minimum, shall conform to ACI318-2005, Sections 11.8.4 and 11.8.5.

7.8.8 Construction Joints

All construction joints in structural walls shall conform to 6.4 of ACI 318-05 and contact surfaces shall be roughened as in 11.7.9 of ACI 318-05.

7.8.9 Discontinuous Walls

Columns supporting discontinuous structural walls shall be reinforced in accordance with 7.5.4.5.

7.9 Special Structural Walls Constructed using Precast Concrete

Special structural Walls constructed using precast concrete shall satisfy all requirements of 7.7 for cast-in-place special structural walls in addition to 7.14.2 and 7.14.3.

7.10 Structural Diaphragms and Trusses

7.10.1 Scope

Floor and roof slabs acting as structural diaphragms to transmit design actions induced by earthquake ground motions shall be designed in accordance with this section. This section also applies to struts, ties, chords, and collector elements that transmit forces induced by earthquakes, as well as trusses serving as parts of the earthquake force-resisting systems.

7.10.2 Cast-in-place Composite-topping Slab Diaphragms

A composite-topping slab cast in place on a precast floor or roof shall be permitted to be used as a structural diaphragm provided the topping slab is reinforced and its connections are proportioned and detailed to provide for a complete transfer of forces to chords, collector elements, and the lateral-force-resisting system. The surface of the previously hardened concrete on which the topping slab is placed shall be clean, free of laitance, and intentionally roughened.

7.10.3 *Cast-in-place Topping Slab Diaphragms*

A cast-in-place non-composite topping on a precast floor or roof shall be permitted to serve as a structural diaphragm, provided the cast-in-place topping acting alone is proportioned and detailed to resist the design forces.

7.10.4 *Minimum Thickness of Diaphragms*

Concrete slabs and composite topping slabs serving as structural diaphragms used to transmit earthquake forces shall not be less than 50 mm (2 in) thick. Topping slabs placed over precast floor or roof elements, acting as structural diaphragms and not relying on composite action with the precast elements to resist the design seismic forces, shall have thickness not less than 65 mm (2.5 in.).

7.10.5 *Reinforcement*

7.10.5.1 The minimum reinforcement ratio for structural diaphragms shall be in conformance with ACI 318-05 Section 7.12. Reinforcement spacing each way in non post-tensioned floor or roof systems shall not exceed 450 mm (18 in). Where welded wire reinforcement is used as the distributed reinforcement to resist shear in topping slabs placed over precast floor and roof elements, the wires parallel to the span of the precast elements shall be spaced not less than 250 mm (10 in) on center. Reinforcement provided for shear strength shall be continuous and shall be distributed uniformly across the shear plane.

7.10.5.2 Bonded tendons used as primary reinforcement in diaphragm chords or collectors shall be proportioned such that the stress due to design seismic forces does not exceed 420 MPa (60,000 psi). Pre-compression from unbonded tendons shall be permitted to resist diaphragm design forces if a complete load path is provided.

7.10.5.3 Structural truss elements, struts, ties, diaphragm chords, and collector elements with compressive stresses exceeding $0.2f'_c$ at any section shall have transverse reinforcement, as given in 7.5.4.1 through 7.5.4.3, over the length of the element. The special transverse reinforcement is permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$. Where design forces have been amplified to account for the overstrength of the vertical elements of the seismic-force-resisting system, the limit of $0.2f'_c$ shall be increased to $0.5f'_c$, and the limit of $0.15f'_c$ shall be increased to $0.4f'_c$.

7.10.5.4 All continuous reinforcement in diaphragms, trusses, struts, ties, chords, and collector elements shall be developed or spliced for f_y in tension.

7.10.6 *Design Forces*

The seismic design forces for structural diaphragms shall be obtained from the lateral load analysis in accordance with the design load combinations.

7.10.7 *Shear Strength*

7.10.7.1 V_n of structural diaphragms shall not exceed

$$V_n = A_{cv} \left(0.17\sqrt{f'_c} + \rho_t f_y \right) \quad V_n = A_{cv} \left(2\sqrt{f'_c} + \rho_t f_y \right) \quad (7.10-1)$$

7.10.7.2 V_n of cast-in-place noncomposite topping slab diaphragms on a precast floor or roof shall not exceed

$$V_n = A_{cv} \rho_t f_y \quad (7.10-2)$$

where A_{cv} is based on the thickness of the topping slab. The required web reinforcement shall be distributed uniformly in both directions.

7.10.7.3 Nominal shear strength shall not exceed $0.66 A_{cv} \sqrt{f'_c}$ ($8 A_{cv} \sqrt{f'_c}$) where A_{cv} is the gross area of the diaphragm cross section.

7.10.8 *Boundary Elements of Structural Diaphragms*

7.10.8.1 Boundary elements of structural diaphragms shall be proportioned to resist the sum of the factored axial forces acting in the plane of the diaphragm and the force obtained from dividing M_u at the section by the distance between the boundary elements of the diaphragm at that section.

7.10.8.2 Splices of tension reinforcement in the chords and collector elements of diaphragms shall develop f_y . Welded splices shall conform to 7.3.6.

7.10.8.3 Reinforcement for chords and collectors at splices and anchorage zones shall satisfy either a) or b):

- a) A minimum center-to-center spacing of three longitudinal bar diameters, but not less than 40mm (1.5 in.), and a minimum concrete clear cover of two and one-half longitudinal bar diameters, but not less than 50mm (2 in).; or
- b) Transverse reinforcement as required by ACI 318-05 Section 11.5.6.3, except as required in 7.10.5.3.

7.10.9 All construction joints in diaphragms shall conform to 6.4 of ACI 318-05 and contact surfaces shall be roughened as in 11.7.9 of ACI 318-05.

7.11 Foundations

7.11.1 *Scope*

7.11.1.1 Foundation resisting earthquake induced forces or transferring earthquake induced forces between structure and ground shall comply with 7.11 and other applicable code provisions.

7.11.1.2 The provisions in this section for piles, drilled piers, caisson, slab on grade shall supplement other applicable code design and construction criteria. See 1.1.5 and 1.1.6 of ACI 318-05.

7.11.2 *Footings, Foundation Mats, and Pile Caps*

7.11.2.1 Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

7.11.2.2 Columns designed assuming fixed-end conditions at the foundation shall comply with 7.11.2.1 and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90 degree hooks near the bottom of the foundation with the free-end of the bars oriented towards the center of the column.

7.11.2.3 Columns or boundary elements of special reinforced concrete structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 7.5.4 provided below the top of the footing. This reinforcement shall extend into the footing a distance no less than the smaller of the depth of the footing, mat, or pile cap, or the development length in tension of the longitudinal reinforcement.

7.11.2.4 Where earthquake effects create uplift forces in boundary elements of special reinforced concrete structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat or pile cap to resist the design load combinations, and shall not be less than required by 10.5 of ACI 318-05.

7.11.3 *Grade Beams and Slabs on Grade*

7.11.3.1 Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

7.11.3.2 Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 450 mm (18 in). Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension or 300 mm (12 in).

7.11.3.3 Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the lateral-force-resisting system shall conform to 7.4.

7.11.3.4 Slabs on grade that resist seismic forces from walls or columns that are part of the lateral-force-resisting system shall be designed as structural diaphragms in accordance with 7.9. The design drawings shall clearly state that the slab on grade is a structural diaphragm and part of the lateral-force resisting system.

7.11.4 *Piles, Piers and Caissons*

7.11.4.1 Provisions of 7.11.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

7.11.4.2 Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members.

7.11.4.3 Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least $1.25f_y$ of the bar.

7.11.4.4 Piles, piers, or caissons shall have transverse reinforcement in accordance with 7.5.4 at locations (a) and (b): (a) At the top of the member for at least 5 times the member cross-sectional dimension, but not less than 1800 mm (5 ft.) below the bottom of the pile cap; (b) For the portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in 7.11.4.4(a).

7.11.4.5 Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

7.12 Members not Designated as Part of the Lateral-force-resisting System

7.12.1 Frame members assumed not to contribute to lateral resistance, except two-way slabs without beams, shall be detailed according to 7.12.2 or 7.12.3 depending on the magnitude of moments induced in those members when subjected to the design displacement δ_u . If effects of δ_u are not explicitly checked, it shall be permitted to apply the requirements of 7.12.3. For two-way slabs without beams, slab-column connections shall meet the requirements of 7.12.5.

7.12.2 Where the induced moments and shears under design displacements, δ_u , of 7.12.1 combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of 7.12.2.1, 7.12.2.2, and 7.12.2.3 shall be satisfied. The gravity load combinations of $(1.2D + 1.0L + 0.2S)$ or $0.9D$, whichever is critical, shall be used. The load factor on the live load, L , shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kN/m^2 (100 lb/ft^2).

7.12.2.1 Members with factored gravity axial forces not exceeding $A_g f'_c / 10$ shall satisfy 7.4.2.1. Stirrups shall be spaced not more than $d/2$ throughout the length of the member.

7.12.2.2 Members with factored gravity axial forces exceeding $A_g f'_c / 10$ shall satisfy 7.5.3, 7.5.4.1(c), 7.5.4.3, and 7.5.5. The maximum longitudinal spacing of ties shall be s_o for the full column height. Spacing s_o shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 150 mm (6 in.).

7.12.2.3 Members with factored gravity axial forces exceeding $0.35 P_o$ shall satisfy 7.12.2.2 and the amount of transverse reinforcement provided shall be one-half of that required by 7.5.4.1 but shall not be spaced greater than s_o for the full height of the column.

7.12.3 If the induced moment or shear under design displacements, δ_u , of 7.12.1 exceeds ϕM_n or ϕV_n of the frame member, or if induced moments are not calculated, the conditions of 7.12.3.1, 7.12.3.2, and 7.12.3.3 shall be satisfied.

7.12.3.1 Materials shall satisfy 7.3.4 and 7.3.5. Welded splices shall satisfy 7.3.6.1.

7.12.3.2 Members with factored gravity axial forces not exceeding $A_g f'_c / 10$ shall satisfy 7.4.2.1 and 7.4.4. Stirrups shall be spaced at not more than $d/2$ throughout the length of the member.

7.12.3.3 Members with factored gravity axial forces exceeding $A_g f'_c / 10$ shall satisfy 7.5.3.1, 7.5.4, 7.5.5, and 7.6.2.1.

7.12.4 Precast concrete frame members assumed not to contribute to lateral resistance, including their connections, shall satisfy (a), (b), and (c), in addition to 7.12.1 through 7.12.3: (a) Ties specified in 7.12.2.2 shall be provided over the entire column height, including the depth of the beams; (b) Structural integrity reinforcement, as specified in 16.5 of ACI 318 – 2005, shall be provided; and (c) Bearing length at support of a beam shall be at least 50 mm (2 in) longer than determined from calculations using bearing strength values from 10.17 of ACI 318 – 2005.

7.12.5 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 11.12.3 of ACI 318 – 2005 and providing V_s not less than $0.29 \sqrt{f'_c} b_o d$ ($3.5 \sqrt{f'_c} b_o d$) shall extend at least four times the slab thickness from the face of the support, unless either (a) or (b) is satisfied:

- a) The requirements of 11.12.6 of ACI 318 – 2005 using the design shear V_u and the induced moment transferred between the slab and column under the design displacement;
- b) The design storey drift ratio does not exceed the larger of 0.005 and $[0.035–0.05(V_u / \phi V_c)]$. Design storey drift ratio shall be taken as the larger of the design storey drift ratios of the adjacent stories above and below the slab-column connection. V_c is defined in 11.12.2 ACI 318 – 2005. V_u is the factored shear force on the slab critical section for two-way action, calculated for the load combination $1.2D + 1.0L + 0.2S$. It shall be permitted to reduce the load factor on L to 0.5 in accordance with 9.2.1(a) ACI 318 – 2005.

7.13 Requirements for Intermediate Moment Frames

7.13.1 The requirements of this section apply to intermediate moment frames.

7.13.2 Reinforcement details in a frame member shall satisfy 7.13.4 if the factored axial compressive load, P_u , for the member does not exceed $A_g f'_c / 10$. If P_u is larger, frame reinforcement details shall satisfy 7.13.5 unless the member has spiral reinforcement according to Eq. (10-5) of ACI 318 – 2005. If a two-way slab system without beams is treated as part of a frame resisting E , load effects for earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy 7.13.6.

7.13.3 ϕV_n of beams, columns, and two-way slabs resisting earthquake effect, E , shall not be less than the smaller of (a) and (b): (a) The sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads; (b) The maximum shear obtained from design load combinations that include E , with E assumed to be twice that prescribed by the governing code for earthquake-resistant design.

7.13.4 Beams

7.13.4.1 The positive moment strength at the face of the joint shall be not less than one-third the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength at any section along the length of the member shall be less than one-fifth the maximum moment strength provided at the face of either joint.

7.13.4.2 At both ends of the member, hoops shall be provided over lengths equal $2h$ measured from the face of the supporting member toward midspan. The first hoop shall be located at not more than 50 mm (2 inches) from the face of the supporting member. Spacing of hoops shall not exceed the smallest of (a), (b), (c), and (d): (a) $d/4$; (b) Eight times the diameter of the smallest longitudinal bar enclosed; (c) 24 times the diameter of the hoop bar; (d) 300 mm. (12 inches).

7.13.4.3 Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

7.13.5 Columns

7.13.5.1 Columns shall be spirally reinforced in accordance with 7.10.4 of ACI 318 -2005 or shall conform with 7.13.5.2 through 7.13.5.4. Section 7.13.5.5 shall apply to all columns.

7.13.5.2 At both ends of the member, hoops shall be provided at spacing s_o over a length l_o measured from the joint face. Spacing s_o shall not exceed the smallest of (a), (b), (c), and (d):

- a) Eight times the diameter of the smallest longitudinal bar enclosed,
- b) 24 times the diameter of the hoop bar,
- c) One-half of the smallest cross-sectional dimension of the frame member,
- d) 300 mm (12 in). Length l_o shall not be less than the largest of (e), (f), and (g),

- e) One-sixth of the clear span of the member,
- f) Maximum cross-sectional dimension of the member,
- g) 450 mm (18 in).

7.13.5.3 The first hoop shall be located at not more than $s_o / 2$ from the joint face.

7.13.5.4 Outside the length l_o , spacing of transverse reinforcement shall conform to 7.10 and 11.5.5.1 of ACI 318-05.

7.13.5.5 Joint transverse reinforcement shall conform to 11.11.2 of ACI 318-05.

7.13.6 Two-way Slabs without Beams

7.13.6.1 Factored slab moment at support related to earthquake effect, E , shall be determined for load combinations given in Eq. (9-5) and (9-7) of ACI 318-05. Reinforcement provided to resist M_{slab} shall be placed within the column strip defined in 13.2.1 of ACI 318 – 2005.

7.13.6.2 Reinforcement placed within the effective width specified in ACI 318 – 2005, Section 13.5.3.2 shall resist $\gamma_f M_{slab}$. Effective slab width for exterior and corner connections shall not extend beyond the column face a distance greater than c_t measured perpendicular to the slab span.

7.13.6.3 Not less than one-half of the reinforcement in the column strip at support shall be placed within the effective slab width given in ACI 318 – 2005, Section 13.5.3.2.

7.13.6.4 Not less than one-quarter of the top reinforcement at the support in the column strip shall be continuous throughout the span.

7.13.6.5 Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

7.13.6.6 Not less than one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop f_y at face of support as defined in ACI 318 – 2005, Section 13.6.2.5.

7.13.6.7 At discontinuous edges of the slab all top and bottom reinforcement at support shall be developed at the face of support as defined in ACI 318 – 2005, Section 13.6.2.5.

7.13.6.8 At the critical sections for columns defined in ACI 318 – 2005, Section 11.12.1.2, two-way shear caused by factored gravity loads shall not exceed $0.4 \phi V_c$, where V_c shall be calculated as defined in ACI 318 – 2005, Section 11.12.2.1 for non-prestressed slabs and in ACI 318 – 2005, Section 11.12.2.2 for prestressed slabs. It shall be permitted to waive this requirement if the contribution of the earthquake-induced factored two way shear stress transferred by eccentricity of shear in accordance with ACI 318 – 2005, Section 11.12.6.1 and 11.12.6.2 at the point of maximum stress does not exceed one-half of the stress ϕv_n permitted by ACI 318 – 2005, Section 11.12.6.2.

7.14 Intermediate Precast Structural Walls

7.14.1 The requirements of this section apply to intermediate precast structural walls used to resist forces induced by earthquake motions.

7.14.2 In connections between wall panels, or between wall panels and the foundation, yielding shall be restricted to steel elements or reinforcement.

7.14.3 Elements of the connection that are not designed to yield shall develop at least $1.5 S_y$.

CHAPTER 8**STRUCTURAL STEEL****8.1 Symbols & Notations**

A_b	=	Cross-sectional area of a horizontal boundary element (HBE), mm ² (in ²).
A_c	=	Cross-sectional area of a vertical boundary element (VBE), mm ² (in ²).
A_f	=	Flange area, mm ² (in ²).
A_g	=	Gross area, mm ² (in ²).
A_s	=	Cross sectional area of the structural steel core, mm ² (in ²).
A_w	=	Link-web area, mm ² (in ²).
A_{sc}	=	Area of the yielding segment of steel core, mm ² (in ²).
A_{sh}	=	Minimum area of tie reinforcement, mm ² (in ²).
A_{sp}	=	Horizontal area of the steel plate in composite shear wall, mm ² (in ²)
A_{st}	=	Area of the stiffener, mm ² (in ²).
b	=	Width of compression element as defined in ANSI/AISC 360-05 Section B4.1, mm (in).
b_{cf}	=	Width of column flange, mm (in).
b_f	=	Flange width, mm (in).
b_w	=	Width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear, mm (in).
C_a	=	Ratio of required strength to available strength.
C_d	=	Coefficient relating relative brace stiffness and curvature.
C_d	=	Deflection amplification factor.
C_r	=	Parameter used for determining the approximate fundamental period.
D	=	Dead load due to the weight of the structural elements and permanent features on the building, N (lb).
D	=	Outside diameter of round HSS, mm (in).
d	=	Overall beam depth, mm (in).
d_b	=	Depth of the steel beam, mm (in).
d_c	=	Overall column depth, mm (in).
d_z	=	Overall panel zone depth between continuity plates, mm (in).
E	=	Earthquake load.
E	=	Effect of horizontal and vertical earthquake-induced loads.
E	=	Modulus of elasticity of steel, $E = 200,000$ MPa (29,000,000 psi).
e	=	EBF link length, mm (in).
EI	=	Flexural elastic stiffness of the chord members of the special segment, N-mm ² (lb-in ²).
f'_c	=	Specified compressive strength of concrete, MPa (psi).
F_y	=	Specified minimum yield stress of the type of steel to be used, MPa (psi). As used in the ANSI/AISC 360-05, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point).
F_{yb}	=	F_y of a beam, MPa (psi).
F_{yc}	=	F_y of a column, MPa (psi).
F_{yh}	=	Specified minimum yield stress of the ties, MPa (psi).
F_{ysc}	=	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa (psi).
F_u	=	Specified minimum tensile strength, MPa (psi).
H	=	Height of storey, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between

h	=	the top of floor slabs at each of the levels above and below, mm (in).
h	=	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, mm (in).
h_{cc}	=	Distance between horizontal boundary element centerlines, mm (in).
h_o	=	Cross-sectional dimension of the confined core region in composite columns measured center-to-center of the transverse reinforcement, mm (in).
I	=	Distance between flange centroids, mm (in).
I_c	=	Moment of inertia, mm^4 (in^4).
I_c	=	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, mm^4 (in^4).
K	=	Effective length factor for prismatic member.
L	=	Live load due to occupancy and moveable equipment, kN (lb).
L	=	Distance between VBE centerlines, mm (in).
l	=	Unbraced length of compression or bracing member, mm (in).
L_b	=	Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm (in).
L_b	=	Link length, mm (in).
L_{cf}	=	Clear distance between VBE flanges, mm (in).
L_h	=	Distance between plastic hinge locations, mm (in).
L_p	=	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, mm (in).
L_{pd}	=	Limiting laterally unbraced length for plastic analysis, mm (in).
L_s	=	Length of the special segment, mm (in).
M_a	=	Required flexural strength, using ASD load combinations, N-mm (lb-in).
M_{av}	=	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, N-mm (lb-in)
M_n	=	Nominal flexural strength, N-mm (lb-in).
M_{nc}	=	Nominal flexural strength of the chord member of the special segment, N-mm (lb-in).
M_p	=	Nominal plastic flexural strength, N-mm (lb-in).
M_{pa}	=	Nominal plastic flexural strength modified by axial load, N-mm (lb-in).
M_{pb}	=	Nominal plastic flexural strength of the beam, N-mm (lb-in).
M_{pc}	=	Nominal plastic flexural strength of the column, N-mm (lb-in).
M_r	=	Expected flexural strength, N-mm (lb-in).
M_{uv}	=	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, N-mm (lb-in).
M_u	=	Required flexural strength, using LRFD load combinations, N-mm (lb-in).
$M_{u,exp}$	=	Expected required flexural strength, N-mm (lb-in).
P_a	=	Required axial strength of a column using ASD load combinations, N (lb).
P_{ac}	=	Required compressive strength using ASD load combinations, N (lb).
P_b	=	Required strength of lateral brace at ends of the link, N (lb).
P_c	=	Available axial strength of a column, N (lb).
P_n	=	Nominal axial strength of a column, N (lb).
P_n	=	Nominal compressive strength of the composite column calculated in accordance with the ANSI/AISC 360-05, N (lb).
P_{nc}	=	Nominal axial compressive strength of diagonal members of the special segment, N (lb).
P_{nt}	=	Nominal axial tensile strength of diagonal members of the special segment, N (lb).

P_r	=	Required compressive strength, N (lb).
P_{rc}	=	Required compressive strength using ASD or LRFD load combinations, N (lb).
P_u	=	Required axial strength of a column or a link using LRFD load combinations, N (lb).
P_u	=	Required axial strength of a composite column, N (lb).
P_{uc}	=	Required compressive strength using LRFD load combinations, N (lb).
P_y	=	Nominal axial yield strength of a member, equal to $F_y A_g$, N (lb).
P_{ysc}	=	Axial yield strength of steel core, N (lb).
Q_l	=	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link.
R	=	Seismic response modification coefficient.
r	=	Governing radius of gyration, mm (in).
R_n	=	Nominal strength, N (lb).
R_t	=	Ratio of the expected tensile strength to the specified minimum tensile strength F_u , as related to over strength in material yield stress R_y .
R_u	=	Required strength.
R_v	=	Panel zone nominal shear strength.
R_y	=	Ratio of the expected yield stress to the specified minimum yield stress, F_y .
r_y	=	Radius of gyration about y-axis, mm (in).
s	=	Spacing of transverse reinforcement measured along the longitudinal axis of the structural composite member, mm (in).
t	=	Thickness of element column web or doubler plate, mm (in).
t_{bf}	=	Thickness of beam flange, mm (in).
t_{cf}	=	Thickness of column flange, mm (in).
t_f	=	Thickness of flange, mm (in).
t_{min}	=	Minimum wall thickness of concrete-filled rectangular HSS, mm (in).
t_w	=	Thickness of web, mm (in).
V_a	=	Required shear strength using ASD load combinations, N (lb).
V_n	=	Nominal shear strength of a member, N (lb).
V_{ne}	=	Expected vertical shear strength of the special segment, N (lb).
V_{ns}	=	Nominal shear strength of the steel plate in a composite plate shear wall, N (lb)
V_p	=	Nominal shear strength of an active link, N (lb).
V_{pa}	=	Nominal shear strength of an active link modified by the axial load magnitude, N (lb).
V_u	=	Required shear strength using LRFD load combinations, N (lb).
w_z	=	Width of panel zone between column flanges, mm (in).
x	=	Parameter used for determining the approximate fundamental period.
Y_{con}	=	Distance from top of steel beam to top of concrete slab or encasement, mm (in).
Y_{PNA}	=	Maximum distance from the maximum concrete compression fiber to the plastic neutral axis, mm (in).
Z	=	Plastic section modulus of a member, mm^3 (in^3).
Z_b	=	Plastic section modulus of the beam, mm^3 (in^3).
z_b	=	Minimum plastic section modulus at the reduced beam section, mm^3 (in^3).
Z_c	=	Plastic section modulus of the column, mm^3 (in^3).
Z_x	=	Plastic section modulus x-axis, mm^3 (in^3).
Z_{RBS}	=	Minimum plastic section modulus at the reduced beam section, mm^3 (in^3).
$\sum M_{pc}^*$	=	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g , from the top and bottom of the beam moment connection.
$\sum M_{pb}^*$	=	Moment at the intersection of the beam and column centerlines determined by

projecting the beam maximum developed moments from the column face.
Maximum developed moments shall be determined from test results.

β	=	Compression strength adjustment factor.
Δ	=	Design storey drift.
Ω	=	Safety factor.
Ω_b	=	Safety factor for flexure = 1.67.
Ω_c	=	Safety factor for compression = 1.67.
Ω_o	=	Horizontal seismic overstrength factor.
Ω_v	=	Safety factor for shear strength of panel zone of beam-to-column connections.
α	=	Angle of diagonal members with the horizontal.
α'	=	Angle of web yielding in radians, as measured relative to the vertical.
ρ'	=	Ratio of required axial force P_u to required shear strength V_u of a link.
λ_p, λ_{ps}	=	Limiting slenderness parameter for compact element Resistance factor.
Φ	=	Resistance factor for flexure.
Φ_b	=	Resistance factor for compression.
Φ_c	=	Resistance factor for shear strength of panel zone of beam-to-column connections.
Φ_v	=	Resistance factor for shear.
ω	=	Strain hardening adjustment factor.

Division I — Structural Steel Buildings

8.2 Definitions

1. Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
2. Terms designated with * are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, design flexural strength.
3. Terms designated with ** are usually qualified by the type of component, for example, web local buckling, flange local bending.

Adjusted brace strength. Strength of a brace in a buckling-restrained braced frame at deformations corresponding to 2.0 times the design storey drift.

Allowable strength*†. Nominal strength divided by the safety factor, R_n / Ω .

Applicable building code (ABC) †. Building code under which the structure is designed.

Amplified seismic load. Horizontal component of earthquake load E multiplied by Ω_o , where E and the horizontal component of E are specified in the applicable building code.

Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this standard.

Available strength*†. Design strength or allowable strength, as appropriate.

ASD (Allowable Strength Design). Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD load combination†. Load combination in the applicable building code intended for allowable strength design (allowable stress design).

Buckling-restrained braced frame (BRBF). Diagonally braced frame satisfying the requirements of Section 8.16 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the design storey drift.

Buckling-restraining system. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the design storey drift.

Casing. Element that resists forces transverse to the axis of the brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.

Column base. Assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

Continuity plates. Column stiffeners at the top and bottom of the panel zone; also known as

transverse stiffeners.

Contractor. Fabricator or erector, as applicable.

Demand critical weld. Weld so designated by these Provisions.

Design earthquake. The earthquake represented by the design response spectrum as specified in the applicable building code.

Design storey drift. Amplified storey drift (drift under the design earthquake, including the effects of inelastic action), determined as specified in the applicable building code.

Design strength*†. Resistance factor multiplied by the nominal strength, ϕR_n .

Diagonal bracing. Inclined structural members carrying primarily axial load that are employed to enable a structural frame to act as a truss to resist lateral loads.

Dual system. Structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear, and concrete or steel shear walls, or steel braced frames (EBF, SCBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-thickness limitations of Table 8.2. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

Eccentrically braced frame (EBF). Diagonally braced frame meeting the requirements of Section 8.15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.

Exempted column. Column not meeting the requirements of Equation 8.9-3 for SMF. Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g .

Expected tensile strength*. Tensile strength of a member, equal to the specified minimum tensile strength, F_u , multiplied by R_t .

Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F_y , multiplied by R_y .

Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section 8.10.

Interstorey drift angle. Interstorey displacement divided by storey height, radians.

Inverted-V-braced frame. See V-braced frame.

k-area. The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC “*k*” dimension) a distance of 38 mm (1½ in) into the web beyond the “*k*” dimension.

K-braced frame. A bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

Lateral bracing member. Member that is designed to inhibit lateral buckling or lateral-torsional buckling of primary framing members.

Link. In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the link is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.

Link rotation angle. Inelastic angle between the link and the beam outside of the link when the total storey drift is equal to the design storey drift.

Link shear design strength. Lesser of the available shear strength of the link developed from the moment or shear strength of the link.

Lowest Anticipated Service Temperature (LAST). The lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (Load and Resistance Factor Design)†. Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD Load Combination†. Load combination in the applicable building code intended for strength design (load and resistance factor design).

Measured flexural resistance. Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix S of ANSI/AISC 341-05.

Nominal load†. Magnitude of the load specified by the applicable building code.

Nominal strength*†. Strength of a structure or component (without the resistance factor or safety factor applied) to resist the load effects, as determined in accordance with the ANSI/AISC 360-05.

Ordinary concentrically braced frame (OCBF). Diagonally braced frame meeting the requirements of Section 8.14 in which all members of the bracing system are subjected primarily to axial forces.

Ordinary moment frame (OMF). Moment frame system that meets the requirements of Section 8.11.

Overstrength factor, Ω_o . Factor specified by the applicable building code in order to determine the amplified seismic load, where required by these Provisions.

Prequalified connection. Connection that complies with the requirements of Appendix P of ANSI/AISC 341-05 or ANSI/AISC 358.

Protected zone. Area of members in which limitations apply to fabrication and attachments. See Section 8.7.4.

Prototype. The connection or brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

Provisions. Refers to sections of this Chapter and applicable provisions of ANSI/AISC 341-05.

Quality assurance plan. Written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.

Reduced beam section. Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.

Required strength*†. Forces, stresses, and deformations produced in a structural component, determined by either structural analysis, for the *LRFD* or *ASD* load combinations, as appropriate, or as specified by the ANSI/AISC 360-05 and these Provisions.

Resistance factor, Φ †. Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Safety factor, Ω †. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic design category. Classification assigned to a building by the applicable building code based upon its seismic use group and the design spectral response acceleration coefficients.

Seismic load resisting system (SLRS). Assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

Seismic response modification coefficient, R . Factor that reduces seismic load effects to strength level as specified by the applicable building code.

Seismic use group. Classification assigned to a structure based on its use as specified by the applicable building code.

Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section 8.13 in which all members of the bracing system are subjected primarily to axial forces.

Special moment frame (SMF). Moment frame system that meets the requirements of Section 8.9.

Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section 8.17

Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section 8.12.

Specification. Refers to the AISC Specification for Structural Steel Buildings, ANSI/AISC 360-05.

Static yield strength. Strength of a structural member or connection determined on the basis of

testing conducted under slow monotonic loading until failure.

Steel core. Axial-force-resisting element of braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

Tested connection. Connection that complies with the requirements of Appendix S of ANSI/AISC 341-05.

V-braced frame. Concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an inverted-V-braced frame.

X-braced frame. Concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-braced frame. Eccentrically braced frame (EBF) in which the stem of the Y is the link of the EBF system.

8.3 Scope

The Seismic Provisions for Structural Steel Buildings, hereinafter referred to as these Provisions, shall govern the design, fabrication and erection of structural steel members and connections in the seismic load resisting systems (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements. These Provisions shall apply when the seismic response modification coefficient, R , (as specified in the Chapter 5, Table 5.13) is taken greater than 3, regardless of the seismic design category. When the seismic response modification coefficient, R , is taken as 3 or less, the structure is not required to satisfy these Provisions, unless specifically required by the applicable building code.

These Provisions shall be applied in conjunction with the AISC Specification for Structural Steel Buildings ANSI/AISC 360-05, hereinafter referred to as the Specification. Members and connections of the SLRS shall satisfy the requirements of the applicable building code, ANSI/AISC 360-05, and these Provisions.

Division I also includes Appendices P, Q, R, S, T, W and X of ANSI/AISC 341-05.

8.4 Loads, Load Combinations, and Nominal Strengths

8.4.1 Loads and Load Combinations

The loads and load combinations shall be as stipulated by the Chapter 5. Where amplified seismic loads are required by these Provisions, the horizontal portion of the earthquake load E (as defined in the applicable building code) shall be multiplied by the overstrength factor, Ω_o , prescribed by Chapter 5 (Table 5.13).

8.4.2 Nominal Strength

The nominal strength of systems, members and connections shall comply with ANSI/AISC 360-

05, except as modified throughout these Provisions.

8.5 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings

8.5.1 *Structural Design Drawings and Specifications*

Structural design drawings and specifications shall show the work to be performed, and include items required by ANSI/AISC 360-05 and the following, as applicable:

1. Designation of the seismic load resisting system (SLRS).
2. Designation of the members and connections that are part of the SLRS.
3. Configuration of the connections.
4. Connection material specifications and sizes.
5. Locations of demand critical welds.
6. Lowest anticipated service temperature (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 10°C (50°F) or higher.
7. Locations and dimensions of protected zones.
8. Locations where gusset plates are to be detailed to accommodate inelastic rotation.
9. Welding requirements as specified in Appendix W, Section W2.1 of Seismic Provisions for Structural Steel buildings ANSI/AISC 341-05.

8.5.2 *Shop Drawings*

Shop drawings shall include items required by ANSI/AISC 360-05 and the following, as applicable:

1. Designation of the members and connections that are part of the SLRS.
2. Connection material specifications.
3. Locations of demand critical shop welds.
4. Locations and dimensions of protected zones.
5. Gusset plates drawn to scale when they are detailed to accommodate inelastic rotation.
6. Welding requirements as specified in Appendix W, Section W2.2 of Seismic Provisions for Structural Steel buildings ANSI/AISC 341-05.

8.5.3 *Erection Drawings*

Erection drawings shall include items required by ANSI/AISC 360-05 and the following, as applicable:

1. Designation of the members and connections that are part of the SLRS.
2. Field connection material specifications and sizes.
3. Locations of demand critical field welds.
4. Locations and dimensions of protected zones.
5. Locations of pretensioned bolts.
6. Field welding requirements as specified in Appendix W, Section W2.3 of Seismic Provisions for Structural Steel buildings ANSI/AISC 341-05.

8.6 Materials

8.6.1 Material Specifications

Structural steel used in the seismic load resisting system (SLRS) shall meet the requirements of ANSI/AISC 360-05 Section A3.1a, except as modified in these Provisions. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 345 MPa (50,000 psi) for systems defined in Sections 8.9, 8.10, 8.12, 8.13, 8.15, 8.16, and 8.17 nor 380 MPa (55,000 psi) for systems defined in Sections 8.11 and 8.14, unless the suitability of the material is determined by testing or other rational criteria.

This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the column base. The structural steel used in the SLRS described in Sections 8.9, 8.10, 8.11, 8.12, 8.13, 8.14, 8.15, 8.16 and 8.17 shall meet one of the following ASTM Specifications: A36/ A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M [Grade 42 (290), 50 (345) or 55 (380)], A588/A588M, A913/A913M [Grade 50(345), 60 (415) or 65 (450)], A992/A992M, or A1011 HSLAS Grade 55 (380). The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

Other steels and non-steel materials in buckling-restrained braced frames are permitted to be used subject to the requirements of Section 8.16 and Appendix T of ANSI/AISC 341-05.

8.6.2 Material Properties for Determination of Required Strength of Members and Connections

When required in these Provisions, the required strength of an element (a member or a connection) shall be determined from the expected yield stress, $R_y F_y$, of an adjoining member, where F_y is the specified minimum yield stress of the grade of steel to be used in the adjoining members and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y , of that material. The available strength of the element, ϕR_n for LRFD and R_n / Ω for ASD, shall be equal to or greater than the required strength, where R_n is the nominal strength of the connection. The expected tensile strength, $R_t F_u$, and the expected yield stress, $R_y F_y$, are permitted to be used in lieu of F_u and F_y , respectively, in determining the nominal strength, R_n , of rupture and yielding limit states within the same member for which the required strength is determined.

The values of R_y and R_t for various steels are given in Table 8.1. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens similar in size and source conducted in accordance with the requirements for the specified grade of steel.

Structural steel, not manufactured in accordance with the ASTM Standards, but meeting the minimum requirements for mechanical properties as per relevant ASTM Standards are allowed for structural use.

8.6.3 Heavy Section CVN Requirements

For structural steel in the SLRS, in addition to the requirements of ANSI/AISC 360-05 Section A3.1c, hot rolled shapes with flanges 38 mm (1½ in) thick and thicker shall have a minimum Charpy V-Notch toughness of 27 J (20 ft-lb) at 21°C (70°F), tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 50 mm (2 in) thick and thicker shall have a minimum Charpy V-Notch toughness of 27 J (20 ft-lb) at 21°C (70°F), measured at any location permitted by ASTM A673, where the plate is used in the following:

1. Members built-up from plate.
2. Connection plates where inelastic strain under seismic loading is expected.
3. As the steel core of buckling-restrained braces.

8.7 Connections, Joints and Fasteners

8.7.1 Scope

Connections, joints and fasteners that are part of the seismic load resisting system (SLRS) shall comply with ANSI/AISC 360-05 Chapter J, and with the additional requirements of this Section. The design of connections for a member that is a part of the SLRS shall be configured such that a ductile limit state in either the connection or the member controls the design.

8.7.2 Bolted Joints

All bolts shall be pretensioned high strength bolts and shall meet the requirements for slip-critical faying surfaces in accordance with ANSI/AISC 360-05 Section J3.8 with a Class A surface. Bolts shall be installed in standard holes or in shortslopped holes perpendicular to the applied load. For brace diagonals, oversized holes shall be permitted when the connection is designed as a slip-critical joint, and the oversized hole is in one ply only.

Alternative hole types are permitted if designated in the Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications (ANSI/AISC 358), or if otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or if determined in a program of qualification testing in accordance with Appendix S or T of ANSI/AISC 341-05. The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with ANSI/AISC 360-05 Sections J3.7 and J3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

Exception: The faying surfaces for end plate moment connections are permitted to be coated with coatings not tested for slip resistance, or with coatings with a slip coefficient less than that of a Class A faying surface. Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

8.7.3 Welded Joints

Welding shall be performed in accordance with Appendix W of ANSI/AISC 341-05. Welding shall be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1 and approved by the engineer of record. The WPS variables shall be within the parameters established by the filler metal manufacturer.

8.7.3.1 General requirements

All welds used in members and connections in the SLRS shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 27 J (20 ft-lb) at minus 18°C (0°F), as determined by the appropriate AWS A5 classification test method or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these Provisions.

8.7.3.2 Demand critical welds

Where welds are designated as demand critical, they shall be made with a filler metal capable of providing a minimum Charpy V-Notch (CVN) toughness of 27 J (20 ft-lb) at -29°C (-20°F) as determined by the appropriate AWS classification test method or manufacturer certification, and 54 J (40 ft-lb) at 21°C (70 °F) as determined by Appendix X of ANSI/AISC 341-05 or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 10°C (50°F) or higher. For structures with service temperatures lower than 10°C (50°F), the qualification temperature for Appendix X of ANSI/AISC 341-05 shall be 11°C (20°F) above the lowest anticipated service temperature, or at a lower temperature.

SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 27 J (20 ft-lb) at a temperature not exceeding -29°C (-20°F) as determined by AWS classification test methods. The manufacturer's certificate of compliance shall be considered sufficient evidence of meeting this requirement.

8.7.4 Protected Zone

Where a protected zone is designated by these Provisions or ANSI/AISC 358, it shall comply with the following:

1. Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the engineer of record.
2. Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Decking arc spot welds as required to secure decking shall be permitted.
3. Welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception: Welded shear studs and other connections shall be permitted when designated in the Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications (ANSI/AISC 358), or as otherwise determined in accordance with a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

Outside the protected zone, calculations based upon the expected moment shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

8.7.5 Continuity Plates and Stiffeners

Corners of continuity plates and stiffeners placed in the webs of rolled shapes shall be clipped as described below. Along the web, the clip shall be detailed so that the clip extends a distance of at least 38 mm (1½ in) beyond the published k detail dimension for the rolled shape. Along the flange, the clip shall be detailed so that the clip does not exceed a distance of 12 mm (½ in) beyond the published k₁ detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. If a curved clip is used, it shall have a minimum radius of 12 mm (½ in).

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified by the engineer of record that they be removed, weld tabs shall not be removed when used in this location.

8.8 Members

8.8.1 Scope

Members in the seismic load resisting system (SLRS) shall comply with ANSI/AISC 360-05 and Section 8.8. For columns that are not part of the SLRS, see Section 8.8.4.2.

8.8.2 Classification of Sections for Local Buckling

8.8.2.1 Compact

When required by these Provisions, members of the SLRS shall have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_p , from ANSI/AISC 360-05 Table B4.1.

8.8.2.2 Seismically Compact

When required by these Provisions, members of the SLRS must have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_{ps} , from Table 8.2.

8.8.3 Column Strength

When $P_u / \Phi_c P_n$ (LRFD) > 0.4 or $\Omega_c P_a / P_n$ (ASD) > 0.4, as appropriate, without consideration of the amplified seismic load,

Where:

Φ_c	= 0.90 (LRFD)	Ω_c = 1.67 (ASD)
P_a	= required axial strength of a column using ASD load combinations, N (lb).	
P_n	= nominal axial strength of a column, N (lb).	
P_u	= required axial strength of a column using LRFD load combinations, N (lb).	

The following requirements shall be met:

1. The required axial compressive and tensile strength, considered in the absence of any applied moment, shall be determined using the load combinations stipulated by the applicable building code including the amplified seismic load.
2. The required axial compressive and tensile strength shall not exceed either of the following:
 - a. The maximum load transferred to the column considering $1.1R_y$ (LRFD) or $(1.1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting beam or brace elements of the building.
 - b. The limit as determined from the resistance of the foundation to overturning uplift.

8.8.4 *Column Splices***8.8.4.1** *General*

The required strength of column splices in the seismic load resisting system (*SLRS*) shall equal the required strength of the columns, including that determined from Sections 8.8.3, 8.9.9, 8.10.9, 8.11.9, 8.13.5 and 8.16.5.2.

In addition, welded column splices that are subject to a calculated net tensile load effect determined using the load combinations stipulated by the applicable building code including the amplified seismic load, shall satisfy both of the following requirements:

1. The available strength of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200 percent of the required strength.
2. The available strength for each flange splice shall be at least equal to $0.5 R_y F_y A_f$ (LRFD) or $(0.5/1.5)R_y F_y A_f$ (ASD), as appropriate, where $R_y F_y$ is the expected yield stress of the column material and A_f is the flange area of the smaller column connected.

Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where PJP groove welded joints are used.

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In moment frames using bolted splices, plates or channels shall be used on both sides of the column web.

The centerline of column splices made with fillet welds or partial-joint-penetration groove welds shall be located 1.2 m (4 ft) or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m (8 ft), splices shall be at half the clear height.

8.8.4.2 *Columns not part of the seismic load resisting system*

Splices of columns that are not a part of the SLRS shall satisfy the following:

- (1) Splices shall be located 1.2 m (4 ft) or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m (8 ft), splices shall be at half the clear height.
- (2) The required shear strength of column splices with respect to both orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/1.5H$ (ASD), as appropriate, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the storey height.

8.8.5 *Column Bases*

The required strength of column bases shall be calculated in accordance with Sections 8.8.5.1, 8.8.5.2, and 8.8.5.3. The available strength of anchor rods shall be determined in accordance with ANSI/AISC 360-05 Section J3.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with ACI 318-05, Appendix D.

Exception: The special requirements in ACI 318-05, Appendix D, for “regions of moderate or high seismic risk, or for structures assigned to intermediate or high

seismic performance or design categories" need not be applied.

8.8.5.1 *Required axial strength*

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

8.8.5.2 *Required shear strength*

The required shear strength of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

1. For diagonal bracing, the horizontal component shall be determined from the required strength of bracing connections for the seismic load resisting system (SLRS).
2. For columns, the horizontal component shall be at least equal to the lesser of the following:
 - a. $2R_y F_y Z_x / H$ (LRFD) or $(2/1.5) R_y F_y Z_x / H$ (ASD), as appropriate, of the column

Where:

H = height of storey, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, in (mm).

- b. The shear calculated using the load combinations of the applicable building code, including the amplified seismic load.

8.8.5.3 *Required flexural strength*

The required flexural strength of column bases, including their attachment to the foundation, shall be the summation of the required strengths of the steel elements that are connected to the column base as follows:

1. For diagonal bracing, the required flexural strength shall be at least equal to the required strength of bracing connections for the SLRS.
2. For columns, the required flexural strength shall be at least equal to the lesser of the following:
 - a. $1.1R_y F_y Z$ (LRFD) or $(1.1/1.5)R_y F_y Z$ (ASD), as appropriate, of the column; or
 - b. The moment calculated using the load combinations of the applicable building code, including the amplified seismic load.

8.8.6 *H-Piles*

8.8.6.1 *Design of H-Piles*

Design of H-piles shall comply with the provisions of the ANSI/AISC 360-05 regarding design of members subjected to combined loads. H-piles shall meet the requirements of Section 8.8.2.2.

8.8.6.2 *Battered H-Piles*

If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.

8.8.6.3 *Tension in H-Piles*

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile. Directly below the bottom of the pile cap, each pile shall be free of attachments and welds for a length at least equal to the depth of the pile cross section.

8.9 Special Moment Frames (SMF)

8.9.1 *Scope*

Special moment frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SMF shall satisfy the requirements in this Section.

8.9.2 *Beam-to-Column Connections*

8.9.2.1 *Requirements*

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the following three requirements:

1. The connection shall be capable of sustaining an interstorey drift angle of at least 0.04 radians.
2. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstorey drift angle of 0.04 radians.
3. The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_y M_p]/L_h \quad (8.9-1)$$

Where:

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y .

M_p = nominal plastic flexural strength, N-mm (lb-in).

L_h = distance between plastic hinge locations, mm (in).

When E as defined in Equation 8.9-1 is used in ASD load combinations that are additive with other transient loads and that are based on Chapter 5, the 0.75 combination factor for transient loads shall not be applied to E .

Connections that accommodate the required interstorey drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

8.9.2.2 *Conformance demonstration*

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 8.9.2.1 by one of the following:

- a. Use of SMF connections designed in accordance with ANSI/AISC 358.
- b. Use of a connection prequalified for SMF in accordance with Appendix P of ANSI/AISC 341-05.
- c. Provision of qualifying cyclic test results in accordance with Appendix S of ANSI/AISC 341-05.

Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

- (i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix S of ANSI/AISC 341-05.
- (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S of ANSI/AISC 341-05.

8.9.2.3 *Welds*

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 8.7.3.2

8.9.2.4 *Protected zones*

The region at each end of the beam subject to inelastic straining shall be designated as a protected zone, and shall meet the requirements of Section 8.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.9.3 *Panel Zone of Beam-to-column Connection (beam web parallel to column web)*

8.9.3.1 *Shear strength*

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or prequalified connection. As a minimum, the required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\phi_v R_v$ and the allowable shear strength shall be R_v/Ω_v where:

$$\phi_v = 1.0 \quad (\text{LRFD}) \qquad \Omega_v = 1.50 \quad (\text{ASD})$$

and the nominal shear strength, R_v , according to the limit state of shear yielding, is determined as specified in ANSI/AISC 360-05 Section J10.6.

8.9.3.2 *Panel zone thickness*

The individual thicknesses, t , of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \quad (8.9-2)$$

Where:

- t = thickness of column web or doubler plate, mm (in)
 d_z = panel zone depth between continuity plates, mm (in)
 w_z = panel zone width between column flanges, mm (in)

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, the total panel zone thickness shall satisfy Equation 8.9-2.

8.9.3.3 *Panel zone doubler plates*

Doubler plates shall be welded to the column flanges using either a complete-joint-penetration groove-welded or fillet-welded joint that develops the available shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

8.9.4 *Beam and Column Limitations*

The requirements of Section 8.8.1 shall be satisfied, in addition to the following.

8.9.4.1 *Width-thickness limitations*

Beam and column members shall meet the requirements of Section 8.8.2.2, unless otherwise qualified by tests.

8.9.4.2 *Beam flanges*

Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.9.5 *Continuity Plates*

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.9.6 Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\Sigma M_{pc}^*}{\Sigma M_{pb}^*} > 1.0 \quad (8.9-3)$$

Where:

ΣM_{pc}^* = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM_{pc}^* is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\Sigma M_{pc}^* = \sum Z_c (F_{yc} - P_{uc}/A_g)$ (LRFD) or $\Sigma Z_c [(F_{yc}/1.5) - P_{ac}/A_g]$ (ASD), as appropriate. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

ΣM_{pb}^* = the sum of the moments in the beams at the intersection of the beam and column centerlines. ΣM_{pb}^* is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\Sigma M_{pb}^* = \sum (1.1R_y F_{yb} Z_b + M_{uv})$ (LRFD) or $\sum [(1.1/1.5)R_y F_{yb} Z_b + M_{av}]$ (ASD), as appropriate. Alternatively, it is permitted to determine ΣM_{pb}^* consistent with a prequalified connection design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05. When connections with reduced beam sections are used, it is permitted to take $\Sigma M_{pb}^* = \sum (1.1R_y F_{yb} Z_{RBS} + M_{uv})$ (LRFD) or $\sum [(1.1/1.5)R_y F_{yb} Z_{RBS} + M_{av}]$ (ASD), as appropriate.

A_g	=	gross area of column, mm ² (in ²).
F_{yc}	=	specified minimum yield stress of column, MPa (psi).
M_{av}	=	the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, N-mm (lb-in).
M_{uv}	=	the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on LRFD load combinations, N-mm (lb-in).
P_{ac}	=	required compressive strength using ASD load combinations, (a positive number) N (lb).
P_{uc}	=	required compressive strength using LRFD load combinations, (a positive number) N (lb).
Z_b	=	plastic section modulus of the beam, mm ³ (in ³).
Z_c	=	plastic section modulus of the column, mm ³ (in ³).
Z_{RBS}	=	minimum plastic section modulus at the reduced beam section, mm ³ (in ³).

Exception: This requirement does not apply if either of the following two conditions is satisfied:

- (a) Columns with $P_{rc} < 0.3P_c$ for all load combinations other than those determined using the amplified seismic load that satisfy either of the

following:

- (i) Columns used in a one-storey building or the top storey of a multistorey building.
- (ii) Columns where: (1) the sum of the available shear strengths of all exempted columns in the storey is less than 20 percent of the sum of the available shear strengths of all moment frame columns in the storey acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that storey is less than 33 percent of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

Where:

For design according to ANSI/AISC 360-05 Section B3.3 (LRFD),

$$P_c = F_{yc} A_g, \text{ N (lb)}$$

P_{rc} = P_{uc} , required compressive strength, using LRFD load combinations, N (lb)

For design according to ANSI/AISC 360-05 Section B3.4 (ASD),

$$P_c = F_{yc} A_g / 1.5, \text{ N (lb)}$$

P_{rc} = P_{ac} , required compressive strength, using ASD load combinations, N (lb)

- (b) Columns in any storey that has a ratio of available shear strength to required shear strength that is 50 percent greater than the storey above.

8.9.7 *Lateral Bracing at Beam-to-Column Connections*

8.9.7.1 *Braced connections*

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 8.9-3 is greater than 2.0.

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

1. The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.
2. Each column-flange lateral brace shall be designed for a required strength that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf} / 1.5$ (ASD), as appropriate.

8.9.7.2 *Unbraced connections*

A column containing a beam-to-column connection with no lateral bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the seismic frame and shall conform to ANSI/AISC 360-05 Chapter H, except that:

1. The required column strength shall be determined from the appropriate load combinations in the applicable building code, except that E shall be taken as the lesser of:
 - (a) The amplified seismic load.
 - (b) 125 percent of the frame available strength based upon either the beam available flexural strength or panel zone available shear strength.
2. The slenderness L/r for the column shall not exceed 60.
3. The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section 8.9.7.1(2) in addition to the second-order moment due to the resulting column flange displacement.

8.9.8 *Lateral Bracing of Beams*

Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = 0.086r_y E/F_y$. Braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the ANSI/AISC 360-05, where $M_r = M_u = R_y ZF_y$ (LRFD) or $M_r = M_a = R_y ZF_y /1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05 or in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_a = 0.06 M_a / h_o$ (ASD), as appropriate, where h_o is the distance between flange centroids; and the required stiffness shall meet the provisions of Equation A-6-8 of Appendix 6 of the ANSI/AISC 360-05 .

8.9.9 *Column Splices*

Column splices shall comply with the requirements of Section 8.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint penetration groove welds that meet the requirements of Section 8.7.3.2. Weld tabs shall be removed. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x /1.5$ (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to $\sum M_{pc} / H$ (LRFD) or $\sum M_{pc} / 1.5H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception: The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses.

8.10 Intermediate Moment Frames (IMF)

8.10.1 *Scope*

Intermediate moment frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. IMF shall meet the requirements in this Section.

8.10.2 Beam-to-Column Connections

8.10.2.1 Requirements

Beam-to-column connections used in the seismic load resisting system (SLRS) shall satisfy the requirements of Section 8.9.2.1, with the following exceptions:

1. The required interstorey drift angle shall be a minimum of 0.02 radian.
2. The required strength in shear shall be determined as specified in Section 8.9.2.1, except that a lesser value of V_u or V_a , as appropriate, is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the applicable building code using the amplified seismic load.

8.10.2.2 Conformance demonstration

Conformance demonstration shall be as described in Section 8.9.2.2 to satisfy the requirements of Section 8.10.2.1 for IMF, except that a connection prequalified for IMF in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.10.2.3 Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 8.7.3.2.

8.10.2.4 Protected zone

The region at each end of the beam subject to inelastic straining shall be treated as a protected zone, and shall meet the requirements of Section 8.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.10.3 Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond ANSI/AISC 360-05.

8.10.4 Beam and Column Limitations

The requirements of Section 8.8.1 shall be satisfied, in addition to the following.

8.10.4.1 Width-thickness limitations

Beam and column members shall meet the requirements of Section 8.8.2.1, unless otherwise qualified by tests.

8.10.4.2 Beam flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. Drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.10.5 Continuity Plates

Continuity plates shall be provided to be consistent with the prequalified connections designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix P of ANSI/AISC 341-05, or as determined in a program of qualification testing in accordance with Appendix S of ANSI/AISC 341-05.

8.10.6 Column-Beam Moment Ratio

No additional requirements beyond the ANSI/AISC 360-05.

8.10.7 Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the ANSI/AISC 360-05.

8.10.8 Lateral Bracing of Beams

Both flanges shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.17r_y E/F_y$. Braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the ANSI/AISC 360-05, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam, and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated loads, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. Where the design is based upon assemblies tested in accordance with Appendix S of ANSI/AISC 341-05, the placement of lateral bracing for the beams shall be consistent with that used in the tests or as required for prequalification in Appendix P of ANSI/AISC 341-05. The required strength of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u / h_o$ (LRFD) or $P_a = 0.06 M_a / h_o$ (ASD), as appropriate, where h_o = distance between flange centroids; and the required stiffness shall meet the provisions of Equation A-6-8 of Appendix 6 of the ANSI/AISC 360-05 .

8.10.9 Column Splices

Column splices shall comply with the requirements of Section 8.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 8.7.3.2.

8.11 Ordinary Moment Frames (OMF)**8.11.1 Scope**

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of

the design earthquake. OMF shall meet the requirements of this Section. Connections in conformance with Sections 8.9.2.2 and 8.9.5 or Sections 8.10.2.2 and 8.10.5 shall be permitted for use in OMF without meeting the requirements of Sections 8.11.2.1, 8.11.2.3, and 8.11.5.

8.11.2 Beam-to-Column Connections

Beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections as follows.

8.11.2.1 Requirements for FR moment connections

FR moment connections that are part of the seismic load resisting system (SLRS) shall be designed for a required flexural strength that is equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, of the beam or girder, or the maximum moment that can be developed by the system, whichever is less. FR connections shall meet the following requirements:

1. Where steel backing is used in connections with complete-joint-penetration (CJP) beam flange groove welds, steel backing and tabs shall be removed, except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:
 - (i) Following the removal of backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 8 mm (5/16 in).
 - (ii) Weld tab removal shall extend to within 3 mm (1/8 in) of the base metal surface, except at continuity plates where removal to within 6 mm (1/4 in) of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of $13 \mu\text{m}$ ($500 \mu\text{in}$) or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 2 mm (1/16 in) below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.
2. Where weld access holes are provided, they shall be as shown in Fig. 8.1. The weld access hole shall have a surface roughness value not to exceed $13 \mu\text{m}$ ($500 \mu\text{in}$), and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the engineer of record. Weld access holes are prohibited in the beam web adjacent to the end-plate in bolted moment end-plate connections.
3. The required strength of double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be $1.1R_y F_y A_g$ (LRFD) or $(1.1/1.5)R_y F_y A_g$ (ASD), as appropriate, of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.
4. For FR moment connections, the required shear strength, V_u or V_a , as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect E :

$$E = 2[1.1R_y M_p]/L_h \quad (8.11-1)$$

where this E is used in ASD load combinations that are additive with other transient loads and that are based on Chapter 5, the 0.75 combination factor for transient loads shall not

be applied to E .

Alternatively, a lesser value of V_u or V_a is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the applicable building code using the amplified seismic load.

8.11.2.2 Requirements for PR moment connections

PR moment connections are permitted when the following requirements are met:

1. Such connections shall be designed for the required strength as specified in Section 8.11.2.1 above.
2. The nominal flexural strength of the connection, M_n , shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.
3. The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.
4. For PR moment connections, V_u or V_a , as appropriate, shall be determined from the load combination above plus the shear resulting from the maximum end moment that the connection is capable of resisting.

8.11.2.3 Welds

Complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be demand critical welds as described in Section 8.7.3.2.

8.11.3 Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the ANSI/AISC 360-05.

8.11.4 Beam and Column Limitations

No requirements beyond Section 8.8.1.

8.11.5 Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity plates shall be provided in accordance with Section J10 of the ANSI/AISC 360-05. Continuity plates shall also be required when:

$$t_{cf} < 0.54 \sqrt{b_f t_{bf} F_{yb} / F_{yc}}$$

or when

$$t_{cf} < b_f / 6$$

Where continuity plates are required, the thickness of the plates shall be determined as follows:

- a. For one-sided connections, continuity plate thickness shall be at least one half of the thickness of the beam flange.
- b. For two-sided connections the continuity plates shall be at least equal in thickness to the thicker of the beam flanges.

The welded joints of the continuity plates to the column flanges shall be made with

either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column flange. The required strength of the welded joints of the continuity plates to the column web shall be the least of the following:

- a. The sum of the available strengths at the connections of the continuity plate to the column flanges.
- b. The available shear strength of the contact area of the plate with the column web.
- c. The weld available strength that develops the available shear strength of the column panel zone.
- d. The actual force transmitted by the stiffener.

8.11.6 *Column-Beam Moment Ratio*

No requirements.

8.11.7 *Lateral Bracing at Beam-to-Column Connections*

No additional requirements beyond the ANSI/AISC 360-05.

8.11.8 *Lateral Bracing of Beams*

No additional requirements beyond the ANSI/AISC 360-05.

8.11.9 *Column Splices*

Column splices shall comply with the requirements of Section 8.8.4.1.

8.12 Special Truss Moment Frames (STMF)

8.12.1 *Scope*

Special truss moment frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the design earthquake. STMF shall be limited to span lengths between columns not to exceed 20 m (65 ft) and overall depth not to exceed 1.8 m (6 ft). The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

8.12.2 *Special Segment*

Each horizontal truss that is part of the seismic load resisting system (SLRS) shall have a special segment that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X- braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a required strength equal to 0.25 times

the nominal tensile strength of the diagonal member. Bolted connections shall not be used for web members within the special segment. Diagonal web members within the special segment shall be made of flat bars of identical sections.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. The required axial strength of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate. The special segment shall be a protected zone meeting the requirements of Section 8.7.4.

8.12.3 Strength of Special Segment Members

The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the required vertical shear strength. The required axial strength in the chord members, determined according to the limit state of tensile yielding, shall not exceed 0.45 times ΦP_n (LRFD) or P_n / Ω (ASD), as appropriate,

$$\Phi = 0.90 \quad (\text{LRFD}) \qquad \Omega = 1.67 \quad (\text{ASD})$$

Where:

$$P_n = F_y A_g$$

The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength, in tension, of the web member, $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

8.12.4 Strength of Non-Special Segment Members

Members and connections of STMF, except those in the special segment specified in Section 8.12.2, shall have a required strength based on the appropriate load combinations in the applicable building code, replacing the earthquake load term E with the lateral loads necessary to develop the expected vertical shear strength of the special segment V_{ne} (LRFD) or $V_{ne} / 1.5$ (ASD), as appropriate, at mid-length, given as:

$$V_{ne} = \frac{3.75R_y M_{nc}}{L_s} + 0.075EI \frac{(L - L_s)}{L_s^3} + R_y (P_{nt} + 0.3P_{nc}) \sin\alpha \quad (8.12-1)$$

Where:

M_{nc} = nominal flexural strength of a chord member of the special segment, N-mm (lb-in).

EI = flexural elastic stiffness of a chord member of the special segment, N-mm²(lb-in²).

L = span length of the truss, mm (in).

L_s = length of the special segment, mm (in).

P_{nt} = nominal tensile strength of a diagonal member of the special segment, N(lb).

P_{nc} = nominal compressive strength of a diagonal member of the special segment, N(lb).

α = angle of diagonal members with the horizontal.

8.12.5 Width-Thickness Limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Section 8.8.2.2.

8.12.6 Lateral Bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of the special segment, and at intervals not to exceed L_p according to ANSI/AISC 360-05 Chapter F, along the entire length of the truss. The required strength of each lateral brace at the ends of and within the special segment shall be:

$$\begin{aligned} P_u &= 0.06 R_y P_{nc} \text{ (LRFD) or} \\ P_a &= (0.06/1.5) R_y P_{nc} \text{ (ASD), as appropriate,} \end{aligned}$$

where P_{nc} is the nominal compressive strength of the special segment chord member. Lateral braces outside of the special segment shall have a required strength of:

$$\begin{aligned} P_u &= 0.02 R_y P_{nc} \text{ (LRFD) or} \\ P_a &= (0.02/1.5) R_y P_{nc} \text{ (ASD), as appropriate.} \end{aligned}$$

The required brace stiffness shall meet the provisions of Equation A-6-4 of Appendix 6 of the ANSI/AISC 360-05, where:

$$\begin{aligned} P_r &= P_u = R_y P_{nc} \text{ (LRFD) or} \\ P_r &= P_a = R_y P_{nc}/1.5 \text{ (ASD), as appropriate.} \end{aligned}$$

8.13 Special Concentrically Braced Frames (SCBF)**8.13.1 Scope**

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. SCBF shall meet the requirements in this Section.

8.13.2 Members**8.13.2.1 Slenderness**

Bracing members shall have $Kl/r < 4\sqrt{EF_y}$

Exception: Braces with $4\sqrt{E/F_y} \leq Kl/r = 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or $(1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

8.13.2.2 Required strength

Where the effective net area of bracing members is less than the gross area, the required tensile strength of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

- a. The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- b. The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

8.13.2.3 Lateral force distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the application of the appropriate load combinations stipulated by the applicable building code including the amplified seismic load. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of bracing.

8.13.2.4 Width-thickness limitations

Column and brace members shall meet the requirements of Section 8.8.2.2.

8.13.2.5 Built-up members

The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member. The sum of the available shear strengths of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Bolted stitches shall not be located within the middle one-fourth of the clear brace length. Exception: Where the buckling of braces about their critical buckling axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

8.13.3 Required Strength of Bracing Connections

8.13.3.1 Required tensile strength

The required tensile strength of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

- a. The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.
- b. The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

8.13.3.2 Required flexural strength

The required flexural strength of bracing connections shall be equal to $1.1 R_y M_p$ (LRFD) or $(1.1/1.5) R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

Exception: Brace connections that meet the requirements of Section 8.13.3.1 and can accommodate the inelastic rotations associated with brace post-buckling deformations need not meet this requirement.

8.13.3.3 Required compressive strength

Bracing connections shall be designed for a required compressive strength based on buckling limit states that is at least equal to $1.1R_yP_n$ (LRFD) or $(1.1/1.5)R_yP_n$ (ASD), as appropriate, where P_n is the nominal compressive strength of the brace.

8.13.4 Special Bracing Configuration Requirements

8.13.4.1 V-Type and inverted-V-Type bracing

V-type and inverted V-type SCBF shall meet the following requirements:

1. The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the applicable building code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the beam shall be determined as follows:
 - a. The forces in all braces in tension shall be assumed to be equal to $R_yF_yA_g$.
 - b. The forces in all adjoining braces in compression shall be assumed to be equal to $0.3P_n$.
2. Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equation A-1-7 and A-1-8 of Appendix 1 of the ANSI/AISC 360-05. Lateral braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the ANSI/AISC 360-05, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

8.13.4.2 K-Type bracing

K-type braced frames are not permitted for SCBF.

8.13.5 Column Splices

In addition to meeting the requirements in Section 8.8.4, column splices in SCBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members. The required shear strength shall be $\sum M_{pc}/H$ (LRFD) or $\sum M_{pc}/1.5H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

8.13.6 Protected Zone

The protected zone of bracing members in SCBF shall include the center one-quarter of the brace length, and a zone adjacent to each connection equal to the brace depth in the plane of buckling. The protected zone of SCBF shall include elements that connect braces to beams and columns and shall satisfy the requirements of Section 8.7.4.

8.14 Ordinary Concentrically Braced Frames (OCBF)

8.14.1 Scope

Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. OCBF shall meet the requirements in this Section. OCBF above the isolation system in seismically isolated structures shall meet the requirements of Sections 8.14.4 and 8.14.5 and need not meet the requirements of Sections 8.14.2 and 8.14.3.

8.14.2 Bracing Members

Bracing members shall meet the requirements of Section 8.8.2.2.

Exception: HSS braces that are filled with concrete need not comply with this provision. Bracing members in K, V, or inverted-V configurations shall have $Kl/r < 4\sqrt{EF_y}$.

8.14.3 Special Bracing Configuration Requirements

Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-column connection and shall meet the following requirements:

1. The required strength shall be determined based on the load combinations of the applicable building code assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E , on the member shall be determined as follows:
 - a. The forces in braces in tension shall be assumed to be equal to $R_y F_y A_g$. For V-type and inverted V-type OCBF, the forces in braces in tension need not exceed the maximum force that can be developed by the system.
 - b. The forces in braces in compression shall be assumed to be equal to $0.3 P_n$.
2. Both flanges shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equations A-1-7 and A-1-8 of Appendix 1 of the ANSI/AISC 360-05. Lateral braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the ANSI/AISC 360-05, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the bracing, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

8.14.4 Bracing Connections

The required strength of bracing connections shall be determined as follows.

1. For the limit state of bolt slip, the required strength of bracing connections shall be determined using the load combinations stipulated by the applicable building code, not including the amplified seismic load.
2. For other limit states, the required strength of bracing connections is the expected yield strength, in tension, of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g / 1.5$ (ASD), as appropriate.

Exception: The required strength of the brace connection need not exceed either of the following:

- a. The maximum force that can be developed by the system
- b. A load effect based upon using the amplified seismic load

8.14.5 *OCBF above Seismic Isolation Systems*

8.14.5.1 *Bracing members*

Bracing members shall meet the requirements of Section 8.8.2.1 and shall have $KI/r < 4\sqrt{E/F_y}$.

8.14.5.2 *K-Type bracing*

K-type braced frames are not permitted.

8.14.5.3 *V-Type and inverted-V-Type bracing*

Beams in V-type and inverted V-type bracing shall be continuous between columns.

8.15 Eccentrically Braced Frames (EBF)

8.15.1 *Scope*

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the links when subjected to the forces resulting from the motions of the design earthquake. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five storeys in height, the upper storey of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the applicable building code. EBF shall meet the requirements in this Section.

8.15.2 *Links*

8.15.2.1 *Limitations*

Links shall meet the requirements of Section 8.8.2.2.

The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

8.15.2.2 *Shear strength*

Except as limited below, the link design shear strength, $\Phi_v V_n$, and the allowable shear strength, V_n/Ω_v , according to the limit state of shear yielding shall be determined as follows:

$$\begin{aligned}V_n &= \text{nominal shear strength of the link, equal to the lesser of } V_p \text{ or } 2M_p/e, \text{ N (lb)} \\ \Phi_v &= 0.90 \text{ (LRFD)} \\ \Omega_v &= 1.67 \text{ (ASD)}\end{aligned}$$

Where:

$$\begin{aligned} M_p &= F_y Z, \text{ N-mm (lb-in)} \\ V_p &= 0.6F_y A_w, \text{ N (lb)} \\ e &= \text{link length, mm (in)} \\ A_w &= (d-2t_f) t_w \end{aligned}$$

The effect of axial force on the link available shear strength need not be considered if:

$$\begin{aligned} P_u &\leq 0.15P_y \quad (\text{LRFD}) \\ \text{or} \\ P_a &\leq (0.15/1.5)P_y \quad (\text{ASD}), \text{ as appropriate.} \end{aligned}$$

Where:

$$\begin{aligned} P_u &= \text{required axial strength using LRFD load combinations, N (lb)} \\ P_a &= \text{required axial strength using ASD load combinations, N (lb)} \\ P_y &= \text{nominal axial yield strength} = F_y A_g, \text{ N (lb)} \end{aligned}$$

If $P_u > 0.15P_y$ (LRFD)

or

$P_a > (0.15/1.5)P_y$ (ASD), as appropriate, the following additional requirements shall be met:

1. The available shear strength of the link shall be the lesser of $\Phi_v V_{pa}$ and $2 \Phi_v M_{pa}/e$ (LRFD)

or V_{pa}/Ω_v and $2(M_{pa}/e)/\Omega_v$ (ASD), as appropriate, where:

$$\Phi_v = 0.90 \quad (\text{LRFD}) \quad \Omega_v = 1.67 \quad (\text{ASD})$$

$$V_{pa} = V_p \sqrt{\left(1 - \left(\frac{P_r}{P_c}\right)^2\right)} \quad (8.15-1)$$

$$M_{pa} = 1.18 M_p \left(1 - \frac{P_r}{P_c}\right) \quad (8.15-2)$$

$P_r = P_u$ (LRFD) or P_a (ASD), as appropriate

$P_c = P_y$ (LRFD) or $P_y/1.5$ (ASD), as appropriate

2. The length of the link shall not exceed:

$$(a) [1.15 - 0.5\rho'(A_w/A_g)] 1.6 M_p / V_p \text{ when } \rho'(A_w/A_g) \geq 0.3 \quad (8.15-3)$$

nor

$$(b) 1.6 M_p / V_p \text{ when } \rho'(A_w/A_g) < 0.3 \quad (8.15-4)$$

Where:

$$\begin{aligned} A_w &= (d-2t_f) t_w \\ \rho' &= P_r / V_r \end{aligned}$$

Where:

$$V_r = V_u \text{ (LRFD) or } V_a \text{ (ASD), as appropriate}$$

- V_u = required shear strength based on LRFD load combinations, N (lb)
 V_a = required shear strength based on ASD load combinations, N (lb)

8.15.2.3 Link rotation angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total storey drift is equal to the design storey drift, Δ . The link rotation angle shall not exceed the following values:

- a. 0.08 radians for links of length $1.6M_p/V_p$ or less.
- b. 0.02 radians for links of length $2.6M_p/V_p$ or greater.
- c. The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

8.15.3 Link Stiffeners

Full-depth web stiffeners shall be provided on both sides of the *link* web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm (3/8 in), whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively. Links shall be provided with intermediate web stiffeners as follows:

- a. Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 radian or $(52t_w - d/5)$ for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.
- b. Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.
- c. Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.
- d. Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.
- e. Intermediate web stiffeners shall be full depth. For links that are less than 635 mm (25 in) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm (3/8 in), whichever is larger, and the width shall be not less than $(b_f/2) - t_w$. For links that are 635 mm (25 in) in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y / 1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y/4$ (LRFD) or $A_{st}F_y/4(1.5)$ (ASD).

8.15.4 Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum *link rotation angle* based on the length of the link, as specified in Section 8.15.2.3. The strength of the connection measured at the column face shall equal at least the nominal shear strength of the link, V_n , as specified in Section 8.15.2.2 at the maximum link rotation angle.

Link-to-column connections shall satisfy the above requirements by one of the following:

- a. Use a connection prequalified for EBF in accordance with Appendix P of ANSI/AISC

- 341-05.
- b. Provide qualifying cyclic test results in accordance with Appendix S of ANSI/AISC 341-05. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
- (i) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Appendix S of ANSI/AISC 341-05.
 - (ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix S of ANSI/AISC 341-05.

Exception: Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened link as described in Section 8.15.6. Full depth stiffeners as required in Section 8.15.3 shall be placed at the link-to-reinforcement interface.

8.15.5 *Lateral Bracing of Link*

Lateral bracing shall be provided at both the top and bottom link flanges at the ends of the link. The required strength of each lateral brace at the ends of the link shall be $P_b = 0.06 M_r/h_o$, where h_o is the distance between flange centroids in mm (in).

For design according to ANSI/AISC 360-05 Section B3.3 (LRFD)
 $M_r = M_{u,exp} = R_y Z F_y$

For design according to ANSI/AISC 360-05 Section B3.4 (ASD)
 $M_r = M_{u,exp}/1.5$

The required brace stiffness shall meet the provisions of Equation A-6-8 of the ANSI/AISC 360-05, where M_r is defined above, $C_d = 1$, and L_b is the link length.

8.15.6 *Diagonal Brace and Beam Outside of Link*

8.15.6.1 *Diagonal brace*

The required combined axial and flexural strength of the diagonal brace shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E , where Q_1 is defined as the axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link $R_y V_n$, where V_n is as defined in Section 8.15.2.2. The available strength of the diagonal brace shall comply with ANSI/AISC 360-05 Chapter H.

Brace members shall meet the requirements of Section 8.8.2.1.

8.15.6.2 *Beam outside link*

The required combined axial and flexural strength of the beam outside of the link shall be

determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term E where Q_1 is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link, $R_y V_n$, where V_n is as defined in Section 8.15.2.2. The available strength of the beam outside of the link shall be determined by the ANSI/AISC 360-05, multiplied by R_y .

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

8.15.6.3 *Bracing connections*

The required strength of the diagonal brace connections, at both ends of the brace, shall be at least equal to the required strength of the diagonal brace, as defined in Section 8.15.6.1. The diagonal brace connections shall also satisfy the requirements of Section 8.13.3.3.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

8.15.7 *Beam-to-Column Connections*

If the EBF system factors in the applicable building code require moment resisting connections away from the link, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in Sections 8.11.2 and 8.11.5.

If the EBF system factors in the applicable building code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

8.15.8 *Required Strength of Columns*

In addition to the requirements in Section 8.8.3, the required strength of columns shall be determined from load combinations as stipulated by the applicable building code, except that the seismic load E shall be the forces generated by 1.1 times the expected nominal shear strength of all links above the level under consideration. The expected nominal shear strength of a link is $R_y V_n$, where V_n is as defined in Section 8.15.2.2.

Column members shall meet the requirements of Section 8.8.2.2.

8.15.9 *Protected Zone*

Links in EBFs are a protected zone, and shall satisfy the requirements of Section 8.7.4. Welding on links is permitted for attachment of link stiffeners, as required in Section 8.15.3.

8.15.10 *Demand Critical Welds*

Complete-joint-penetration groove welds attaching the link flanges and the link web to the column are demand critical welds, and shall satisfy the requirements of Section 8.7.3.2.

8.16. Buckling-Restrained Braced Frames (BRBF)

8.16.1 Scope

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the design earthquake. BRBF shall meet the requirements in this Section. Where the applicable building code does not contain design coefficients for BRBF, the provisions of Appendix R of ANSI/AISC 341-05 shall apply.

8.16.2 Bracing Members

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

8.16.2.1 Steel Core

The steel core shall be designed to resist the entire axial force in the brace.

The brace design axial strength, ΦP_{ysc} (LRFD), and the brace allowable axial strength, P_{ysc}/Ω (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$$\begin{aligned} P_{ysc} &= F_{ysc} A_{sc} && \text{(8.16-1)} \\ \Phi &= 0.90 && \text{(LRFD)} \\ \Omega &= 1.67 && \text{(ASD)} \end{aligned}$$

Where:

F_{ysc} = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa (psi).

A_{sc} = net area of steel core, mm² (in²).

Plates used in the steel core that are 50 mm (2 in) thick or greater shall satisfy the minimum notch toughness requirements of Section 8.6.3. Splices in the steel core are not permitted.

8.16.2.2 Buckling-restraining system

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the design storey drift. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design storey drift.

8.16.2.3 Testing

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Appendix T of ANSI/AISC 341-05. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace

sub-assemblage that includes brace connection rotational demands complying with Appendix T of ANSI/AISC 341-05, Section T4 and the other shall be either a uniaxial or a sub-assemblage test complying with Appendix T of ANSI/AISC 341-05, Section T5. Both test types are permitted to be based upon one of the following:

- a. Tests reported in research or documented tests performed for other projects.
- b. Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Appendix T of ANSI/AISC 341-05 are met.

8.16.2.4 Adjusted brace strength

Where required by these Provisions, bracing connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength. The adjusted brace strength in compression shall be $\beta\omega R_y P_{ysc}$. The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

Exception: The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.

The compression strength adjustment factor, β , shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Appendix T, Section T6.3 of ANSI/AISC 341-05 for the range of deformations corresponding to 2.0 times the design storey drift. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.

The strain hardening adjustment factor, ω , shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Appendix T, Section T6.3 of ANSI/AISC 341-05 (for the range of deformations corresponding to 2.0 times the design storey drift) to F_{ysc} of the test specimen. The larger value of β from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

8.16.3 Bracing Connections

8.16.3.1 Required strength

The required strength of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be 1.1 times the adjusted brace strength in compression (LRFD) or 1.1/1.5 times the adjusted brace strength in compression (ASD).

8.16.3.2 Gusset plates

The design of connections shall include considerations of local and overall buckling. Bracing consistent with that used in the tests upon which the design is based is required.

8.16.4 Special Requirements Related to Bracing Configuration

V-type and inverted-V-type braced frames shall meet the following requirements:

1. The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the applicable building code assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E , on the beam shall be determined from the adjusted brace strengths in tension and compression.
2. Beams shall be continuous between columns. Both flanges of beams shall be laterally braced. Lateral braces shall meet the provisions of Equations A-6-7 and A-6-8 of Appendix 6 of the ANSI/AISC 360-05, where $M_r = M_u = R_y ZF_y$ (LRFD) or $M_r = M_a = R_y ZF_y / 1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Section 8.16.4(1). K-type braced frames are not permitted for BRBF.

8.16.5 Beams and Columns

Beams and columns in BRBF shall meet the following requirements.

8.16.5.1 Width-thickness limitations

Beam and column members shall meet the requirements of Section 8.8.2.2.

8.16.5.2 Required strength

The required strength of beams and columns in BRBF shall be determined from load combinations as stipulated in the applicable building code. For load combinations that include earthquake effects, the earthquake effect, E , shall be determined from the adjusted brace strengths in tension and compression.

The required strength of beams and columns need not exceed the maximum force that can be developed by the system.

8.16.5.3 Splices

In addition to meeting the requirements in Section 8.8.4, column splices in BRBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. The required shear strength shall be $\sum M_{pc} / H$ (LRFD) or $\sum M_{pc} / 1.5H$ (ASD), as appropriate, where $\sum M_{pc}$ is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

8.16.6 Protected Zone

The *protected zone* shall include the steel core of bracing members and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section 8.7.4.

8.17 Special Plate Shear Walls (SPSW)

8.17.1 Scope

Special plate shear walls (SPSW) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SPSW shall meet the requirements of this Section. Where the applicable building code does not contain design coefficients for SPSW, the provisions of Appendix R of ANSI/AISC 341-05 shall apply.

8.17.2 Webs

8.17.2.1 Shear strength

The panel design shear strength, ΦV_n (LRFD), and the allowable shear strength, V_n/ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

Where:

$$\begin{aligned} V_n &= 0.42 F_y t_w L_{cf} \sin 2\alpha & (8.17-1) \\ \Phi &= 0.90 & (\text{LRFD}) \\ Q &= 1.67 & (\text{ASD}) \end{aligned}$$

t_w = thickness of the web, mm (in)

L_{cf} = clear distance between VBE flanges, mm (in)

α is the angle of web yielding in radians, as measured relative to the vertical, and it is given by:

$$\tan^4 \alpha = \frac{1 + \frac{t_w L}{2A_c}}{1 + t_w h \left(\frac{1}{A_b} + \frac{h^3}{360 I_c L} \right)} \quad (8.17-2)$$

h = distance between HBE centerlines, mm (in)

A_b = cross-sectional area of a HBE, mm^2 (in^2)

A_c = cross-sectional area of a VBE, mm^2 (in^2)

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, mm^4 (in^4)

L = distance between VBE centerlines, mm (in)

8.17.2.2 Panel aspect ratio

The ratio of panel length to height, L/h , shall be limited to $0.8 < L/h \leq 2.5$.

8.17.2.3 Openings in webs

Openings in webs shall be bounded on all sides by HBE and VBE extending the full width and height of the panel, respectively, unless otherwise justified by testing and analysis.

8.17.3 *Connections of Webs to Boundary Elements*

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α , defined by Equation 8.17-2.

8.17.4 *Horizontal and Vertical Boundary Elements***8.17.4.1** *Required strength*

In addition to the requirements of Section 8.8.3, the required strength of VBE shall be based upon the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α .

The required strength of HBE shall be the greater of the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α or that determined from the load combinations in the applicable building code assuming the web provides no support for gravity loads.

The beam-column moment ratio provisions in Section 8.9.6 shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

8.17.4.2 *HBE-to-VBE connections*

HBE-to-VBE connections shall satisfy the requirements of Section 8.11.2. The required shear strength, V_u , of a HBE-to-VBE connection shall be determined in accordance with the provisions of Section 8.11.2, except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle α .

8.17.4.3 *Width-thickness limitations*

HBE and VBE members shall meet the requirements of Section 8.8.2.2.

8.17.4.4 *Lateral bracing*

HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086r_y E/F_y$. Both flanges of HBE shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE flange nominal strength, $F_y b_f t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Equation A-6-8 of Appendix 6 of the ANSI/AISC 360-05. In these equations, M_r shall be computed as $R_y ZF_y$ (LRFD) or M_r shall be computed as $R_y ZF_y/1.5$ (ASD), as appropriate, and $C_d = 1.0$.

8.17.4.5 *VBE splices*

VBE splices shall comply with the requirements of Section 8.8.4.

8.17.4.6 *Panel zones*

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section 8.9.3.

8.17.4.7 *Stiffness of vertical boundary elements*

The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web I_c , not less than $0.00307 t_w h^4/L$.

8.18 *Quality Assurance Plan*

8.18.1 *Scope*

When required by the applicable building code or the engineer of record, a quality assurance plan shall be provided. The quality assurance plan shall include the requirements of Appendix Q of ANSI/AISC 341-05.

Division II — Composite Structural Steel and Reinforced Concrete Buildings

8.19 Definitions

These terms are in addition to those listed in Division I.

Boundary member. Portion along wall and diaphragm edge strengthened with structural steel sections and/or longitudinal steel reinforcement and transverse reinforcement.

Collector element. Member that serves to transfer loads between floor diaphragms and the members of the seismic load resisting system.

Composite beam. Structural steel beam in contact with and acting compositely with reinforced concrete via bond or shear connectors.

Composite brace. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a brace.

Composite column. Reinforced-concrete-encased structural steel section (rolled or built-up) or concrete-filled steel section used as a column.

Composite eccentrically braced frame (C-EBF). Composite braced frame meeting the requirements of Section 8.32.

Composite intermediate moment frame (C-IMF). Composite moment frame meeting the requirements of Section 8.28.

Composite ordinary braced frame (C-OBF). Composite braced frame meeting the requirements of Section 8.31.

Composite ordinary moment frame (C-OMF). Composite moment frame meeting the requirements of Section 8.29.

Composite partially restrained moment frame (C-PRMF). Composite moment frame meeting the requirements of Section 8.26.

Composite shear wall. Reinforced concrete wall that has unencased or reinforced-concrete-encased structural steel sections as boundary members.

Composite slab. Concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the seismic load resisting system.

Composite special concentrically braced frame (C-CBF). Composite braced frame meeting the requirements of Section 8.30.

Composite special moment frame (C-SMF). Composite moment frame meeting the requirements of Section 8.27.

Composite steel plate shear wall (C-SPW). Wall consisting of steel plate with reinforced concrete encasement on one or both sides that provides out-of-plane stiffening to prevent buckling of the steel plate and meeting the requirements of Section 8.35.

Coupling beam. Structural steel or composite beam connecting adjacent reinforced concrete wall elements so that they act together to resist lateral loads.

Encased composite beam. Composite beam completely enclosed in reinforced concrete.

Encased composite column. Structural steel column (rolled or built-up) completely encased in reinforced concrete.

Face bearing plates. Stiffeners attached to structural steel beams that are embedded in reinforced concrete walls or columns. The plates are located at the face of the reinforced concrete to provide confinement and to transfer loads to the concrete through direct bearing.

Filled composite column. Round or rectangular structural steel section filled with concrete.

Fully composite beam. Composite beam that has a sufficient number of shear connectors to develop the nominal plastic flexural strength of the composite section.

Intermediate seismic systems. Seismic systems designed assuming moderate inelastic action occurs in some members under the design earthquake.

Load-carrying reinforcement. Reinforcement in composite members designed and detailed to resist the required loads.

Ordinary reinforced concrete shear wall with structural steel elements (C-ORCW). Composite shear walls meeting the requirements of Section 8.33.

Ordinary seismic systems. Seismic systems designed assuming limited inelastic action occurs in some members under the design earthquake.

Partially composite beam. Unencased composite beam with a nominal flexural strength controlled by the strength of the shear stud connectors.

Partially restrained composite connection. Partially restrained (PR) connections as defined in the ANSI/AISC 360-05 that connect partially or fully composite beams to steel columns with flexural resistance provided by a force couple achieved with steel reinforcement in the slab and a steel seat angle or similar connection at the bottom flange.

Reinforced-concrete-encased shapes. Structural steel sections encased in reinforced concrete.

Restraining bars. Steel reinforcement in composite members that is not designed to carry required loads, but is provided to facilitate the erection of other steel reinforcement and to provide anchorage for stirrups or ties. Generally, such reinforcement is not spliced to be continuous.

Special reinforced concrete shear walls composite with structural steel elements (C-SRCW). Composite shear walls meeting the requirements of Section 8.34.

Special seismic systems. Seismic systems designed assuming significant inelastic action occurs in some members under the design earthquake.

Unencased composite beam. Composite beam wherein the steel section is not completely enclosed in reinforced concrete and relies on mechanical connectors for composite action with a reinforced slab or slab on metal deck.

8.20 Scope

These Provisions shall govern the design, fabrication, and erection of composite structural steel

and reinforced concrete members and connections in the seismic load resisting systems (SLRS) in buildings and other structures, where other structures are defined as those designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting systems. These provisions shall apply when the seismic response modification coefficient, R , (as specified in the applicable building code) is taken greater than 3. When the seismic response modification coefficient, R , is taken as 3 or less, the structure is not required to satisfy these provisions unless required by the applicable building code.

The requirements of Division II modify and supplement the requirements of Division I and form these Provisions. They shall be applied in conjunction with the AISC Specification for Structural Steel Buildings, ANSI/AISC 360-05, hereinafter referred to as the Specification. The applicable requirements of the Building Code Requirements for Structural Concrete and Commentary, ACI 318-05, as modified in these Provisions shall be used for the design of reinforced concrete components in composite SLRS.

For seismic load resisting systems incorporating reinforced concrete components designed according to ACI 318-05, the requirements for load and resistance factor design as specified in Section B3.3 of the ANSI/AISC 360-05 shall be used. When the design is based upon elastic analysis, the stiffness properties of the component members of composite systems shall reflect their condition at the onset of significant yielding of the structure.

Wherever these Provisions refer to the applicable building code (ABC) and there is no local building code, the loads, load combinations, system limitations and general design requirements shall be those in SEI/ASCE 7.

8.21 General Seismic Design Requirements

The required strength and other provisions for seismic design categories (SDCs) and seismic use groups and the limitations on height and irregularity shall be as specified in the applicable building code. The design storey drift and storey drift limits shall be determined as required in the applicable building code.

8.22 Loads, Load Combinations, and Nominal Strengths

8.22.1 Loads and Load Combinations

Where amplified seismic loads are required by these Provisions, the horizontal portion of the earthquake load E (as defined in the applicable building code) shall be multiplied by the overstrength factor Ω_o prescribed by the applicable building code.

For the seismic load resisting system (SLRS) incorporating reinforced concrete components designed according to ACI 318-05, the requirements of Section B3.3 of the ANSI/AISC 360-05 shall be used.

8.22.2 Nominal Strength

The nominal strength of systems, members, and connections shall be determined in accordance with the requirements of the ANSI/AISC 360-05, except as modified throughout these Provisions.

8.23 Materials

8.23.1 *Structural Steel*

Structural steel members and connections used in composite seismic load resisting systems (SLRS) shall meet the requirements of ANSI/AISC 360-05 Section A3. Structural steel used in the composite SLRS described in Sections 8.26, 8.27, 8.30, 8.32, 8.34 and 8.35 shall also meet the requirements in Division I Sections 8.6 and 8.7.

8.23.2 *Concrete and Steel Reinforcement*

Concrete and steel reinforcement used in composite components in composite SLRS shall meet the requirements of Sections 7.3.4 through 7.3.7.

Exception: Concrete and steel reinforcement used in the composite ordinary seismic systems described in Sections 8.29, 8.31 and 8.33 shall meet the requirements of ANSI/AISC 360-05 Chapter I and ACI 318-05, excluding Chapter 21.

8.24 Composite Members

8.24.1 *Scope*

The design of composite members in the SLRS described in Sections 8.26 through 8.35 shall meet the requirements of this Section and the material requirements of Section 8.23.

8.24.2 *Composite Floor and Roof Slabs*

The design of composite floor and roof slabs shall meet the requirements of ASCE 3. Composite slab diaphragms shall meet the requirements in this Section.

8.24.2.1 *Load transfer*

Details shall be designed so as to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizontal framing system.

8.24.2.2 *Nominal shear strength*

The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318-05 excluding Chapter 22. Alternatively, the composite diaphragm nominal shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

8.24.3 *Composite Beams*

Composite beams shall meet the requirements of ANSI/AISC 360-05 Chapter I. Composite beams that are part of composite-special moment frames (C-SMF) shall also meet the requirements of Section 8.27.3.

8.24.4 *Encased Composite Columns*

This section is applicable to columns that (1) consist of reinforced-concrete- encased shapes with a structural steel area that comprises at least 1 percent of the total composite column cross

section; and (2) meet the additional limitations of ANSI/AISC 360-05 Section I2.1. Such columns shall meet the requirements of ANSI/AISC 360-05 Chapter I, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 8.24.4.2 and 8.24.4.3 shall apply as required in the descriptions of the composite seismic systems in Sections 8.26 through 8.35.

Columns that consist of reinforced-concrete-encased shapes shall meet the requirements for reinforced concrete columns of ACI 318-05 except as modified for:

1. The structural steel section shear connectors in Section 8.24.4.1 (2).
2. The contribution of the reinforced-concrete-encased shape to the strength of the column as provided in ACI 318-05.
3. The seismic requirements for reinforced concrete columns as specified in the description of the composite seismic systems in Sections 8.26 through 8.35.

8.24.4.1 Ordinary seismic system requirements

The following requirements for encased composite columns are applicable to all composite systems, including ordinary seismic systems:

1. The available shear strength of the column shall be determined in accordance with ANSI/AISC 360-05 Section I2.1d. The nominal shear strength of the tie reinforcement shall be determined in accordance with ACI 318-05 Sections 11.5.6.2 through 11.5.6.9. In ACI 318-05 Sections 11.5.6.5 and 11.5.6.9, the dimension b_w shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear.
2. Composite columns designed to share the applied loads between the structural steel section and the reinforced concrete encasement shall have shear connectors that meet the requirements of ANSI/AISC 360-05 Section I2.1.
3. The maximum spacing of transverse ties shall meet the requirements of ANSI/AISC 360-05 Section I2.1.

Transverse ties shall be located vertically within one-half of the tie spacing above the top of the footing or lowest beam or slab in any storey and shall be spaced as provided herein within one-half of the tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter that is not less than one-fiftieth of the greatest side dimension of the composite member, except that ties shall not be smaller than No. 3 bars and need not be larger than No. 5 bars. Alternatively, welded wire fabric of equivalent area is permitted as transverse reinforcement except when prohibited for intermediate and special seismic systems.

4. Load-carrying reinforcement shall meet the detailing and splice requirements of ACI 318-05 Sections 7.8.1 and 12.17. Load-carrying reinforcement shall be provided at every corner of a rectangular cross-section. The maximum spacing of other load carrying or restraining longitudinal reinforcement shall be one-half of the least side dimension of the composite member.
5. Splices and end bearing details for encased composite columns in ordinary seismic systems shall meet the requirements of the ANSI/AISC 360-05 and ACI 318 Section 7.8.2. The design shall comply with Sections 7.3.7 and 7.11 and Section 21.2.6 of ACI 318-05. The design shall consider any adverse behavioral effects due to abrupt changes in either the member stiffness or the nominal tensile strength. Such locations shall include transitions to reinforced concrete sections without embedded structural steel members, transitions to bare structural steel sections, and column bases.

8.24.4.2 *Intermediate seismic system requirements*

Encased composite columns in intermediate seismic systems shall meet the following requirements in addition to those of Section 8.24.4.1:

1. The maximum spacing of transverse bars at the top and bottom shall be the least of the following:
 - (a) one-half the least dimension of the section
 - (b) 8 longitudinal bar diameters
 - (c) 24 tie bar diameters
 - (d) 300 mm(12 in)

These spacings shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

- (a) one-sixth the vertical clear height of the column
- (b) the maximum cross-sectional dimension
- (c) 450 mm (18 in)
2. Tie spacing over the remaining column length shall not exceed twice the spacing defined above.
3. Welded wire fabric is not permitted as transverse reinforcement in intermediate seismic systems.

8.24.4.3 *Special seismic system requirements*

Encased composite columns in special seismic systems shall meet the following requirements in addition to those of Sections 8.24.4.1 and 8.24.4.2:

1. The required axial strength for encased composite columns and splice details shall meet the requirements in Division I Section 8.8.3.
2. Longitudinal load-carrying reinforcement shall meet the requirements of Section 7.5.3.
3. Transverse reinforcement shall be hoop reinforcement as defined in Chapter 7 and shall meet the following requirements:
 - (i) The minimum area of tie reinforcement A_{sh} shall meet the following:

$$A_{sh} = 0.09h_{cc}s \left(1 - \frac{F_y A_s}{P_n}\right) \left(\frac{f'_c}{F_{yh}}\right) \quad (8.24-1)$$

Where:

- h_{cc} = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, mm (in)
- s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm (in)
- F_y = specified minimum yield stress of the structural steel core, MPa (psi)
- A_s = cross-sectional area of the structural core, mm^2 (in^2)
- P_n = nominal compressive strength of the composite column calculated in accordance with the ANSI/AISC 360-05 , N (lb)
- f'_c = specified compressive strength of concrete, MPa (psi)
- F_{yh} = specified minimum yield stress of the ties, MPa (psi). Equation 8.24-1 need not be

satisfied if the nominal strength of the reinforced-concrete-encased structural steel section alone is greater than the load effect from a load combination of $1.0D + 0.5L$.

- (ii) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of six longitudinal load-carrying bar diameters or 150 mm (6 in).
 - (iii) When specified in Sections 8.24.4.3(4), 8.24.4.3(5) or 8.24.4.3(6), the maximum spacing of transverse reinforcement shall be the lesser of one-fourth the least member dimension or 100 mm (4 in). For this reinforcement, cross ties, legs of overlapping hoops, and other confining reinforcement shall be spaced not more than 350 mm(14 in) on center in the transverse direction.
4. Encased composite columns in braced frames with nominal compressive loads that are larger than 0.2 times P_n shall have transverse reinforcement as specified in Section 8.24.4.3(3)(iii) over the total element length. This requirement need not be satisfied if the nominal strength of the reinforced-concrete-encased steel section alone is greater than the load effect from a load combination of $1.0D + 0.5L$.
5. Composite columns supporting reactions from discontinued stiff members, such as walls or braced frames, shall have transverse reinforcement as specified in Section 8.24.4.3(3)(iii) over the full length beneath the level at which the discontinuity occurs if the nominal compressive load exceeds 0.1 times P_n . Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the reinforced- concrete-encased shape and longitudinal reinforcement. This requirement need not be satisfied if the nominal strength of the reinforced-concrete-encased structural steel section alone is greater than the load effect from a load combination of $1.0D + 0.5L$.
6. Encased composite columns used in a C-SMF shall meet the following requirements:
- (i) Transverse reinforcement shall meet the requirements in Section 8.24.4.3(3) at the top and bottom of the column over the region specified in Section 8.24.4.2.
 - (ii) The strong-column/weak-beam design requirements in Section 8.23 shall be satisfied. Column bases shall be detailed to sustain inelastic flexural hinging.
 - (iii) The required shear strength of the column shall meet the requirements of Section 7.5.5.1.
7. When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 300 mm (12 in). When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the reinforced-concrete-encased shape and longitudinal reinforcement.
8. Welded wire fabric is not permitted as transverse reinforcement for special seismic systems.

8.24.5 Filled Composite Columns

This Section is applicable to columns that meet the limitations of ANSI/AISC 360-05 Section I2.2. Such columns shall be designed to meet the requirements of ANSI/AISC 360-05 Chapter I, except as modified in this Section.

1. The nominal shear strength of the composite column shall be the nominal shear strength of the structural steel section alone, based on its effective shear area. The concrete shear capacity may be used in conjunction with the shear strength from the steel shape provided the design includes an appropriate load transferring mechanism.
2. In addition to the requirements of Section 8.24.5(1), in the special seismic systems described in Sections 8.27, 8.30 and 8.32, the design loads and column splices for filled composite columns shall also meet the requirements of Division I Section 8.8.

3. Filled composite columns used in C-SMF shall meet the following requirements in addition to those of Sections 8.24.5(1) and 8.24.5(2):
 - (i) The minimum required shear strength of the column shall meet the requirements in Section 7.5.5.1.
 - (ii) The strong-column/weak-beam design requirements in Section 8.27.5 shall be met. Column bases shall be designed to sustain inelastic flexural hinging.
 - (iii) The minimum wall thickness of concrete-filled rectangular HSS shall be

$$t_{\min} = b \sqrt{F_y / (2E)} \quad (8.24-2)$$

for the flat width b of each face, where b is as defined in ANSI/AISC 360-05 Table B4.1.

8.25 Composite Connections

8.25.1 Scope

This Section is applicable to connections in buildings that utilize composite or dual steel and concrete systems wherein seismic load is transferred between structural steel and reinforced concrete components.

Composite connections shall be demonstrated to have strength, ductility and toughness comparable to that exhibited by similar structural steel or reinforced concrete connections that meet the requirements of Division I and ACI 318-05, respectively. Methods for calculating the connection strength shall meet the requirements in this Section.

8.25.2 General Requirements

Connections shall have adequate deformation capacity to resist the required strength at the design storey drift. Additionally, connections that are required for the lateral stability of the building under seismic loads shall meet the requirements in Sections 8.26 through 8.35 based upon the specific system in which the connection is used. When the available strength of the connected members is based upon nominal material strengths and nominal dimensions, the determination of the available strength of the connection shall account for any effects that result from the increase in the actual nominal strength of the connected member.

8.25.3 Nominal Strength of Connections

The nominal strength of connections in composite structural systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the requirements of Sections 8.25.3(1) through 8.25.3(5).

1. When required, force shall be transferred between structural steel and reinforced concrete through (a) direct bearing of headed shear studs or suitable alternative devices; (b) by other mechanical means; (c) by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer; or (d) by a combination of these means. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism. The contribution of different mechanisms can be combined only if the stiffness and deformation capacity of the mechanisms are compatible.

The nominal bearing and shear-friction strengths shall meet the requirements of ACI 318-05 Chapters 10 and 11. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25 percent for the composite seismic systems described in Sections 8.27, 8.30, 8.32, 8.34 and 8.35.

2. The available strength of structural steel components in composite connections shall be determined in accordance with Division I and the ANSI/AISC 360-05. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out-of-plane buckling. Face bearing plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.
3. The nominal shear strength of reinforced-concrete-encased steel panel- zones in beam-to-column connections shall be calculated as the sum of the nominal strengths of the structural steel and confined reinforced concrete shear elements as determined in Division I Section 8.3 and Section 7.6 respectively.
4. Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with ACI 318-05 Chapter 12. Additionally, development lengths for the systems described in Sections 8.27, 8.30, 8.32, 8.34 and 8.35 shall meet the requirements of Section 7.6.4.
5. Connections shall meet the following additional requirements:
 - (i) When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, braces, and walls.
 - (ii) For connections between structural steel or composite beams and reinforced concrete or encased composite columns, transverse hoop reinforcement shall be provided in the connection region of the column to meet the requirements of Section 7.6, except for the following modifications:
 - (a) Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing plates welded to the beams between the flanges.
 - (b) Lap splices are permitted for perimeter ties when confinement of the splice is provided by face bearing plates or other means that prevents spalling of the concrete cover in the systems described in Sections 8.28, 8.29, 8.31 and 8.33.
 - (c) The longitudinal bar sizes and layout in reinforced concrete and composite columns shall be detailed to minimize slippage of the bars through the beam-to-column connection due to high force transfer associated with the change in column moments over the height of the connection.

8.26 Composite Partially Restrained (PR) Moment Frames (C-PRMF)

8.26.1 Scope

This Section is applicable to frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections that meet the requirements in ANSI/AISC 360-05 Section B3.6b(b). Composite partially restrained moment frames (C-PRMF) shall be designed so that under earthquake loading yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as column base connections. Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength

and drift of C-PRMF.

8.26.2 Columns

Structural steel columns shall meet the requirements of Division I Sections 8.6 and 8.8 and the ANSI/AISC 360-05.

8.26.3 Composite Beams

Composite beams shall be unencased, fully composite and shall meet the requirements of ANSI/AISC 360-05 Chapter I. For purposes of analysis, the stiffness of beams shall be determined with an effective moment of inertia of the composite section.

8.26.4 Moment Connections

The required strength of the beam-to-column PR moment connections shall be determined considering the effects of connection flexibility and second-order moments. In addition, composite connections shall have a nominal strength that is at least equal to 50 percent of M_p , where M_p is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action. Connections shall meet the requirements of Section 8.25 and shall have a total interstorey drift angle of 0.04 radians that is substantiated by cyclic testing as described in Division I Section 8.9.2.2.

8.27 Composite Special Moment Frames (C-SMF)

8.27.1 Scope

This Section is applicable to moment frames that consist of either composite or reinforced concrete columns and either structural steel or composite beams. Composite special moment frames (C-SMF) shall be designed assuming that significant inelastic deformations will occur under the design earthquake, primarily in the beams, but with limited inelastic deformations in the columns and/or connections.

8.27.2 Columns

Composite columns shall meet the requirements for special seismic systems of Sections 8.24.4 or 8.24.5, as appropriate. Reinforced concrete columns shall meet the requirements of Chapter 7, excluding Section 7.11.

8.27.3 Beams

Composite beams that are part of C-SMF shall also meet the following requirements:

1. The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed

$$Y_{PNA} = \frac{Y_{con} + d_b}{1 + \left[\frac{1700 F_y}{E} \right]} \quad (8.27-1)$$

Y_{con} = distance from the top of the steel beam to the top of concrete, mm (in)
 d_b = depth of the steel beam, mm (in)

- F_y = specified minimum yield stress of the steel beam, MPa(psi)
 E = elastic modulus of the steel beam, MPa (psi)
2. Beam flanges shall meet the requirements of Division I Section 8.9.4, except when reinforced-concrete-encased compression elements have a reinforced concrete cover of at least 50 mm (2 in) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements of Section 7.4.3.

Neither structural steel nor composite trusses are permitted as flexural members to resist seismic loads in C-SMF unless it is demonstrated by testing and analysis that the particular system provides adequate ductility and energy dissipation capacity.

8.27.4 Moment Connections

The required strength of beam-to-column moment connections shall be determined from the shear and flexure associated with the expected flexural strength, $R_y M_n$ (LRFD) or $R_y M_n / 1.5$ (ASD), as appropriate, of the beams framing into the connection. The nominal strength of the connection shall meet the requirements in Section 8.25. In addition, the connections shall be capable of sustaining a total interstorey drift angle of 0.04 radian. When beam flanges are interrupted at the connection, the connections shall demonstrate an interstorey drift angle of at least 0.04 radian in cyclic tests that is substantiated by cyclic testing as described in Division I Section 8.9.2.2. For connections to reinforced concrete columns with a beam that is continuous through the column so that welded joints are not required in the flanges and the connection is not otherwise susceptible to premature fractures, the inelastic rotation capacity shall be demonstrated by testing or other substantiating data.

8.27.5 Column-Beam Moment Ratio

The design of reinforced concrete columns shall meet the requirements of Section 7.5.2. The column-to-beam moment ratio of composite columns shall meet the requirements of Division I Section 8.9.6 with the following modifications:

1. The available flexural strength of the composite column shall meet the requirements of ANSI/AISC 360-05 Chapter I with consideration of the required axial strength, P_{rc} .
2. The force limit for Exception (a) in Division I Section 8.9.6 shall be $P_{rc} < 0.1P_c$.
3. Composite columns exempted by the minimum flexural strength requirement in Division I Section 8.9.6(a) shall have transverse reinforcement that meets the requirements in Section 8.24.4.3(3).

8.28 Composite Intermediate Moment Frames (C-IMF)

8.28.1 Scope

This Section is applicable to moment frames that consist of either composite or reinforced concrete columns and either structural steel or composite beams. Composite intermediate moment frames (C-IMF) shall be designed assuming that inelastic deformation under the design earthquake will occur primarily in the beams, but with moderate inelastic deformation in the columns and/or connections.

8.28.2 Columns

Composite columns shall meet the requirements for intermediate seismic systems of Section 8.24.4 or 8.24.5. Reinforced concrete columns shall meet the requirements of Section 7.13.

8.28.3 Beams

Structural steel and composite beams shall meet the requirements of the ANSI/AISC 360-05.

8.28.4 Moment Connections

The *nominal strength* of the connections shall meet the requirements of Section 8.25. The required strength of beam-to-column connections shall meet one of the following requirements:

- a. The required strength of the connection shall be based on the forces associated with plastic hinging of the beams adjacent to the connection.
- b. Connections shall meet the requirements of Section 8.25 and shall demonstrate a total interstorey drift angle of at least 0.03 radian in cyclic tests.

8.29 Composite Ordinary Moment Frames (C-OMF)

8.29.1 Scope

This Section is applicable to moment frames that consist of either composite or reinforced concrete columns and structural steel or composite beams. Composite ordinary moment frames (C-OMF) shall be designed assuming that limited inelastic action will occur under the design earthquake in the beams, columns and/or connections.

8.29.2 Columns

Composite columns shall meet the requirements for ordinary seismic systems in Section 8.24 or 8.24.5, as appropriate. Reinforced concrete columns shall meet the requirements of ACI 318-05, excluding Chapter 21.

8.29.3 Beams

Structural steel and composite beams shall meet the requirements of the ANSI/AISC 360-05.

8.29.4 Moment Connections

Connections shall be designed for the load combinations in accordance with ANSI/AISC 360-05 Sections B3.3 and B3.4, and the available strength of the connections shall meet the requirements in Section 8.25 and Section 8.11.2 of Division I.

8.30 Composite Special Concentrically Braced Frames (C-CBF)

8.30.1 Scope

This Section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design. Columns shall be structural steel, composite structural steel, or reinforced concrete. Beams and braces shall be either structural steel or composite structural steel. Composite special concentrically braced frames (C-CBF) shall be designed assuming that inelastic action under the design earthquake will occur primarily through tension yielding and/or buckling of braces.

8.30.2 *Columns*

Structural steel columns shall meet the requirements of Division I Sections 8.6 and 8.8. Composite columns shall meet the requirements for special seismic systems of Section 8.24.4 or 8.24.5. Reinforced concrete columns shall meet the requirements for structural truss elements of Chapter 7.

8.30.3 *Beams*

Structural steel beams shall meet the requirements for special concentrically braced frames (SCBF) of Division I Section 8.13. Composite beams shall meet the requirements of the ANSI/AISC 360-05 Chapter I and the requirements for special concentrically braced frames (SCBF) of Division I Section 8.13.

8.30.4 *Braces*

Structural steel braces shall meet the requirements for SCBF of Division I Section 8.13. Composite braces shall meet the requirements for composite columns of Section 8.30.

8.30.5 *Connections*

Bracing connections shall meet the requirements of Section 8.25 and Division I Section 8.13.

8.31 Composite Ordinary Braced Frames (C-OBF)

8.31.1 *Scope*

This Section is applicable to concentrically braced frame systems that consist of composite or reinforced concrete columns, structural steel or composite beams, and structural steel or composite braces. Composite ordinary braced frames (C-OBF) shall be designed assuming that limited inelastic action under the design earthquake will occur in the beams, columns, braces, and/or connections.

8.31.2 *Columns*

Encased composite columns shall meet the requirements for ordinary seismic systems of Sections 8.24.4. Filled composite columns shall meet the requirements of Section 8.24.5 for ordinary seismic systems. Reinforced concrete columns shall meet the requirements of ACI 318-05 excluding Chapter 21.

8.31.3 *Beams*

Structural steel and composite beams shall meet the requirements of the ANSI/AISC 360-05 .

8.31.4 *Braces*

Structural steel braces shall meet the requirements of the ANSI/AISC 360-05. Composite braces shall meet the requirements for composite columns of Sections 8.24.4.1, 8.24.5, and 8.31.2.

8.31.5 *Connections*

Connections shall be designed for the load combinations in accordance with ANSI/AISC 360-05 Sections B3.3 and B3.4, and the available strength of the connections shall meet the requirements in Section 8.25.

8.32 Composite Eccentrically Braced Frames (C-EBF)

8.32.1 *Scope*

This Section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and column, or intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace. Composite eccentrically braced frames (C-EBF) shall be designed so that inelastic deformations under the design earthquake will occur only as shear yielding in the links.

Diagonal braces, columns, and beam segments outside of the link shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened link. Columns shall be either composite or reinforced concrete. Braces shall be structural steel. Links shall be structural steel as described in this Section. The available strength of members shall meet the requirements in the ANSI/AISC 360-05, except as modified in this Section. C-EBF shall meet the requirements of Division I Section 8.15, except as modified in this Section.

8.32.2 *Columns*

Reinforced concrete columns shall meet the requirements for structural truss elements of Chapter 7. Composite columns shall meet the requirements for special seismic systems of Sections 8.24.4 or 8.24.5. Additionally, where a link is adjacent to a reinforced concrete column or encased composite column, transverse column reinforcement meeting the requirements of Section 7.5.4 (or Section 8.24.4.3(6)i for composite columns) shall be provided above and below the link connection. All columns shall meet the requirements of Division I Section 8.15.8.

8.32.3 *Links*

Links shall be unencased structural steel and shall meet the requirement for eccentrically braced frame (EBF) links in Division I Section 8.15. It is permitted to encase the portion of the beam outside of the link in reinforced concrete. Beams containing the link are permitted to act compositely with the floor slab using shear connectors along all or any portion of the beam if the composite action is considered when determining the nominal strength of the link.

8.32.4 *Braces*

Structural steel braces shall meet the requirements for EBF of Division I Section 8.15.

8.32.5 *Connections*

In addition to the requirements for EBF of Division I Section 8.15, connections shall meet the requirements of Section 8.25.

8.33 Ordinary Reinforced Concrete Shear Walls Composite with Structural Steel Elements (C-ORCW)**8.33.1 Scope**

The requirements in this Section apply when reinforced concrete walls are composite with structural steel elements, either as infill panels, such as reinforced concrete walls in structural steel frames with unencased or reinforced-concrete- encased structural steel sections that act as boundary members, or as structural steel coupling beams that connect two adjacent reinforced concrete walls. Reinforced concrete walls shall meet the requirements of ACI 318-05 excluding Chapter 21.

8.33.2 Boundary Members

Boundary members shall meet the requirements of this Section:

1. When unencased structural steel sections function as boundary members in reinforced concrete infill panels, the structural steel sections shall meet the requirements of the ANSI/AISC 360-05. The required axial strength of the boundary member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the boundary members in conjunction with the shear wall. The reinforced concrete wall shall meet the requirements of ACI 318-05 excluding Chapter 21.
2. When reinforced-concrete-encased shapes function as boundary members in reinforced concrete infill panels, the analysis shall be based upon a transformed concrete section using elastic material properties. The wall shall meet the requirements of ACI 318-05 excluding Chapter 21. When the reinforced-concrete-encased structural steel boundary member qualifies as a composite column as defined in ANSI/AISC 360-05 Chapter I, it shall be designed to meet the ordinary seismic system requirements of Section 8.24.4.1. Other- wise, it shall be designed as a composite column to meet the requirements of ACI 318-05 Section 10.16 and Chapter I of the ANSI/AISC 360-05.
3. Headed shear studs or welded reinforcement anchors shall be provided to transfer vertical shear forces between the structural steel and reinforced concrete. Headed shear studs, if used, shall meet the requirements of ANSI/AISC 360-05 Chapter I. Welded reinforcement anchors, if used, shall meet the requirements of AWS D1.4.

8.33.3 Steel Coupling Beams

Structural steel coupling beams that are used between two adjacent reinforced concrete walls shall meet the requirements of the ANSI/AISC 360-05 and this Section:

1. Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear that can be generated by the nominal bending and shear strength of the coupling beam. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall boundary member. Connection strength for the transfer of loads between the coupling beam and the wall shall meet the requirements of Section 8.25.
2. Vertical wall reinforcement with nominal axial strength equal to the nominal shear strength of the coupling beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement

placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement.

8.33.4 *Encased Composite Coupling Beams*

Encased composite sections serving as coupling beams shall meet the requirements of Section 8.33.3 as modified in this Section:

1. Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the maximum possible combination of moment and shear capacities of the encased composite steel coupling beam.
2. The nominal shear capacity of the encased composite steel coupling beam shall be used to meet the requirement in Section 8.33.3(1).
3. The stiffness of the encased composite steel coupling beams shall be used for calculating the required strength of the shear wall and coupling beam.

8.34 *Special Reinforced Concrete Shear Walls Composite with Structural Steel Elements (C-SRCW)*

8.34.1 *Scope*

Special reinforced concrete shear walls composite with structural steel elements(C-SRCW) systems shall meet the requirements of Section 8.33 for C-ORCW and the shear-wall requirement of ACI 318-05 and Chapter 7, except as modified in this Section.

8.34.2 *Boundary Members*

In addition to the requirements of Section 8.33(1), unencased structural steel columns shall meet the requirements of Division I Sections 8.6 and 8.8.

In addition to the requirements of Section 8.33.2(2), the requirements in this Section shall apply to walls with reinforced-concrete-encased structural steel boundary members. The wall shall meet the requirements of ACI 318-05 and Chapter 7. Reinforced-concrete-encased structural steel boundary members that qualify as composite columns in ANSI/AISC 360-05 Chapter I shall meet the special seismic system requirements of Section 8.24.4. Otherwise, such members shall be designed as composite compression members to meet the requirements of ACI 318-05 Section 10.16 including the special seismic requirements for boundary members in Section 7.8.6. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of $2h$ into the wall, where h is the overall depth of the boundary member in the plane of the wall.

Headed shear studs or welded reinforcing bar anchors shall be provided as specified in Section 8.33.2(3). For connection to unencased structural steel sections, the nominal strength of welded reinforcing bar anchors shall be reduced by 25 percent from their static yield strength.

8.34.3 *Steel Coupling Beams*

In addition to the requirements of Section 8.33.3, structural steel coupling beams shall meet the requirements of Division I Sections 8.15.2 and 8.15.3. When required in Division I Section 8.15.3, the coupling rotation shall be assumed as 0.08 radian unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the design earthquake. Face bearing plates shall be provided on both sides of the coupling beams at the face of the reinforced

concrete wall. These stiffeners shall meet the detailing requirements of Division I Section 8.15.3. Vertical wall reinforcement as specified in Section 8.33.3(2) shall be confined by transverse reinforcement that meets the requirements for boundary members of Section 7.8.6.

8.34.4 Encased Composite Coupling Beams

Encased composite sections serving as coupling beams shall meet the requirements of Section 8.34.3, except the requirements of Division I Section 8.15.3 need not be met.

8.35 Composite Steel Plate Shear Walls (C-SPW)

8.35.1 Scope

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite boundary members.

8.35.2 Wall Elements

The available shear strength shall be ϕV_{ns} (LRFD) or V_{ns} / Ω (ASD), as appropriate, according to the limit state of shear yielding of composite steel plate shear walls (C-SPW) with a stiffened plate conforming to Section 8.35.2(1) shall be:

$$V_{ns} = 0.6A_{sp}F_y \quad (8.35-1)$$

$$\phi = 0.90 \quad (\text{LRFD}) \quad \Omega = 1.67 \quad (\text{ASD})$$

V_{ns} = nominal shear strength of the steel plate, N (lb)

A_{sp} = horizontal area of stiffened steel plate, mm² (in²)

F_y = specified minimum yield stress of the plate, MPa (psi)

The available shear strength of C-SPW with a plate that does not meet the stiffening requirements in Section 8.35.2(1) shall be based upon the strength of the plate, excluding the strength of the reinforced concrete, and meet the requirements of the ANSI/AISC 360-05 Sections G2 and G3.

1. The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to V_{ns} . The concrete thickness shall be a minimum of 100mm (4 in) on each side when concrete is provided on both sides of the steel plate and 200mm (8 in) when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet or exceed the detailing requirements in ACI 318-05 Section 14.3. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 450mm (18 in).

Seismic forces acting perpendicular to the plane of the wall as specified by the applicable building code shall be considered in the design of the composite wall system.

2. The steel plate shall be continuously connected on all edges to structural steel framing and boundary members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The design of welded and bolted connectors shall meet the additional requirements of Division I Section 8.7.

8.35.3 *Boundary Members*

Structural steel and composite boundary members shall be designed to resist the shear capacity of plate and any reinforced concrete portions of the wall active at the design storey drift. Composite and reinforced concrete boundary members shall also meet the requirements of Section 8.34.2. Steel boundary members shall also meet the requirements of Division I, Section 8.17.

8.35.4 *Openings*

Boundary members shall be provided around openings as required by analysis.

8.36 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings

Structural design drawings and specifications, shop drawings, and erection drawings for composite steel and steel building construction shall meet the requirements of Division I Section 8.5.

For reinforced concrete and composite steel building construction, the contract documents, shop drawings, and erection drawings shall also indicate the following:

- a. Bar placement, cutoffs, lap and mechanical splices, hooks and mechanical anchorages.
- b. Tolerance for placement of ties and other transverse reinforcement.
- c. Provisions for dimensional changes resulting from temperature changes, creep and shrinkage.
- d. Location, magnitude, and sequencing of any prestressing or post-tensioning present.
- e. If concrete floor slabs or slabs on grade serve as diaphragms, connection details between the diaphragm and the main lateral-load resisting system shall be clearly identified.

8.37 Quality Assurance Plan

When required by the Applicable Building Code (ABC) or the engineer of record, a quality assurance plan shall be provided. For the steel portion of the construction, the provisions of Division I, Section 8.18.

Table 8.1 - R_y and R_t Values for Different Member Types

Applications	R_y	R_t
Hot-rolled structural shapes and bars:		
▪ ASTM A36/A36M	1.5	1.2
• ASTM A572/572M Grade 42 (290)	1.3	1.1
• ASTM A572/572M Grade 50 (345) or 55 (380), ASTM A913/A913M Grade 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M, A1011 HSLAS Grade 55 (380)	1.1	1.1
▪ ASTM A529 Grade 50 (345)	1.2	1.2
▪ ASTM A529 Grade 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
▪ ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe:		
▪ ASTM A53/A53M	1.6	1.2
Plates:		
• ASTM A36/A36M	1.3	1.2
▪ ASTM A572/A572M Grade 50 (345), ASTM A588/A588M	1.1	1.2

Table 8.2 - Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios	
		λ_{ps} (seismically compact)	
Un-stiffened Elements	Flexure in flanges of rolled or built-up I-shaped sections [a], [c], [e], [g], [h]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [b], [h]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [d]	b/t	$0.38 \sqrt{E/F_y}$
	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces [c], [g]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in flanges of H-pile sections	b/t	$0.45 \sqrt{E/F_y}$
	Flat bars [f]	b/t	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees [g]	b/t	$0.30 \sqrt{E/F_y}$
	Uniform compression in stems of tees [g]	d/t	$0.30 \sqrt{E/F_y}$

Stiffened Elements	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios	
			λ_{ps} (seismically compact)	
	Webs in flexural compression in Beams in SMF, Section 8.13, unless Noted otherwise Webs in flexural compression in beams in SMF, Section 8.13, unless noted otherwise	h/t_w	2.45	$\sqrt{E/F_y}$
	Webs in flexural compression or combined flexure and axial compression [a], [c], [g], [h], [i], [j]	h/t_w	for $C_a \leq 0.125$	[k]
			3.14	$\sqrt{E/F_y} (1-1.54C_a)$
			for $C_a > 0.125$	[k]
			1.12	$\sqrt{E/F_y} (2.33-C_a) \geq$
			1.49	$\sqrt{E/F_y}$
	Round HSS in axial and/or flexural compression [c], [g]	D/t	0.044	$\sqrt{E/F_y}$
	Rectangular HSS in axial and/or flexural compression [c], [g]	b/t or h/t_w	0.64	$\sqrt{E/F_y}$
	Webs of H.Pile sections	h/t_w	0.94	$\sqrt{E/F_y}$
[a]	Required for beams in SMF, Section 8.9 and SPSW, Section 8.17.			
[b]	Required for columns in SMF, Section 8.9, unless the ratios from Equation 8.9-3 are greater than 2.0 where it is permitted to use λ_p .			
[c]	Required for braces and columns in SCBF, Section 8.13 and braces in OCBF, Section 8.14.			
[d]	It is permitted to use λ_p for columns in STMF, Section 8.12 and columns in EBF, Section 8.15.			
[e]	Required for link in EBF, Section 8.15, except it is permitted to use λ_p for flanges of links of length $1.6M_p/V_p$ or less, where M_p and V_p are defined in Section 8.15.			
[f]	Diagonal web members within the special segment of STMF, Section 8.12.			
[g]	Chord members of STMF, Section 8.12.			
[h]	Required for beams and columns in BRBF, Section 8.16.			
[i]	Required for columns in SPSW, Section 8.17.			
[j]	For columns in STMF, Section 8.12; columns in SMF, if the ratio from Equation 8.9-3 is greater than 2.0; columns in EBF, Section 8.15; or EBF webs of links of length $1.6M_p/V_p$ or less, it is permitted to use the following for λ_p .			
	For $C_a \leq 0.125$, $\lambda_p = 3.76 \sqrt{E/F_y} (1-275C_a)$			
	For $C_a > 0.125$, $\lambda_p = 1.12 \sqrt{E/F_y} (2.33-C_a) \geq 1.49 \sqrt{E/F_y}$			
[k]	For LFRD, $C_a = P_u/\Phi_b P_y$ For ASD, $C_a = \Omega_b P_a/P_y$ Where P_a = required compressive strength (ASD), N (lb) P_u = required compressive strength (LRFD), N (lb). P_y = axial yield strength, N (lb) Φ_b = 0.90 Ω_b = 1.67			

CHAPTER 9

MASONRY

9.1 Symbols and Notations

A	=	Total area of load bearing walls, mm ² (in ²).
A_b	=	Cross-sectional area of anchor bolt, mm ² (in ²).
A_e	=	Effective area of masonry, mm ² (in ²).
A_g	=	Gross area of wall, mm ² (in ²).
A_n	=	Gross area of masonry beam, mm ² (in ²).
A_{jh}	=	Total area of special horizontal reinforcement through wall frame joint, mm ² (in ²).
A_{mv}	=	Net area of masonry section bounded by wall thickness and length of section in direction of shear force considered, mm ² (in ²).
A_p	=	Area of tension (pullout) cone of embedded anchor bolt projected onto surface of masonry, mm ² (in ²).
A_s	=	Effective cross-sectional area of reinforcement in column or flexural member, mm ² (in ²).
A_{se}	=	Effective area of reinforcement, mm ² (in ²).
A_{sh}	=	Total cross-sectional area of rectangular tie reinforcement for confined core, mm ² (in ²).
A_v	=	Area of reinforcement required for shear reinforcement perpendicular to longitudinal reinforcement, mm ² (in ²).
a	=	Depth of equivalent rectangular stress block, mm (in).
B_{sn}	=	Nominal shear strength of anchor bolt, kN (lb).
B_t	=	Allowable tensile force on anchor bolt, kN (lb).
B_{tn}	=	Nominal tensile strength of anchor bolt, kN (lb).
B_v	=	Allowable shear force on anchor bolt, kN (lb).
b	=	Effective width of rectangular member or width of flange for T and I sections, mm (in).
b_{su}	=	Factored shear force supported by anchor bolt, kN (lb).
b_t	=	Computed tensile force on anchor bolt, kN (lb).
b_{tu}	=	Factored tensile force supported by anchor bolt, kN (lb).
b_v	=	Computed shear force on anchor bolt, kN (lb).
b'	=	Width of web in T or I section, mm (in).
C_d	=	Nominal shear strength coefficient as obtained from Table 9.13.
c	=	Distance from neutral axis to extreme fiber, mm (in).
D	=	Dead loads, or related internal moments and forces, kN (lb).
d	=	Distance from compression face of flexural member to centroid of longitudinal tensile reinforcement, mm (in).
d_b	=	Diameter of reinforcing bar, mm (in).
d_{bb}	=	Diameter of largest beam longitudinal reinforcing bar passing through, or anchored in, a joint, mm (in).
d_{bp}	=	Diameter of largest pier longitudinal reinforcing bar passing through a joint, mm (in).
E	=	Load effects of earthquake, or related internal moments and forces.
E_m	=	Modulus of elasticity of masonry, MPa (psi).
E_s	=	Modulus of elasticity of steel, MPa (psi).
e	=	Eccentricity of P_{uf} , mm (in).
e_{mu}	=	Maximum usable compressive strain of masonry.
F_a	=	Allowable average axial compressive stress in columns for centroidally applied axial load only, MPa (psi).

F_b	=	Allowable flexural compressive stress in members subjected to bending load only, MPa (psi).
F_{br}	=	Allowable bearing stress in masonry, MPa (psi).
F_s	=	Allowable stress in reinforcement, MPa (psi).
F_{sc}	=	Allowable compressive stress in column reinforcement, MPa (psi).
F_t	=	Allowable flexural tensile stress in masonry, MPa (psi).
F_v	=	Allowable shear stress in masonry, MPa (psi).
f_a	=	Computed axial compressive stress due to design axial load, MPa (psi).
f_b	=	Computed flexural stress in extreme fiber due to design bending loads only, MPa (psi).
f_{md}	=	Computed compressive stress due to dead load only, MPa (psi).
f_r	=	Modulus of rupture, MPa (psi).
f_s	=	Computed stress in reinforcement due to design loads, MPa (psi).
f_v	=	Computed shear stress due to design load, MPa (psi).
f_y	=	Tensile yield stress of reinforcement, MPa (psi).
f_{yh}	=	Tensile yield stress of horizontal reinforcement, MPa (psi).
f'_g	=	Specified compressive strength of grout at age of 28 days, MPa (psi).
f'_m	=	Specified compressive strength of masonry at age of 28 days, MPa (psi).
G	=	Shear modulus of masonry, MPa (psi).
h	=	Height of wall between points of support, mm (in).
h_b	=	Beam depth, mm (in).
h_c	=	Cross-sectional dimension of grouted core measured center to center of confining reinforcement, mm (in).
h_p	=	Pier depth in plane of wall frame, mm (in).
h'	=	Effective height of wall or column, mm (in).
I	=	Moment of inertia about neutral axis of cross-sectional area, mm ⁴ (in ⁴).
I_g, I_{cr}	=	Gross, cracked moment of inertia of wall cross section, mm ⁴ (in ⁴).
j	=	Ratio or distance between centroid of flexural compressive forces and centroid of tensile forces of depth, d , mm (in).
K	=	Reinforcement cover or clear spacing, whichever is less, mm (in).
k	=	Ratio of depth of compressive stress in flexural member to depth, d .
L	=	Live loads, or related internal moments and forces, kN (lb).
L_w	=	Length of wall, mm (in).
l	=	Length of wall or segment, mm (in).
l_b	=	Embedment depth of anchor bolt, mm (in).
l_{be}	=	Anchor bolt edge distance, the least distance measured from edge of masonry to surface of anchor bolt, mm (in).
l_d	=	Required development length of reinforcement, mm (in).
l_{de}	=	Designed development length of reinforcement, mm (in).
M	=	Design moment, N-mm (lb-in).
M_{cr}	=	Nominal cracking moment strength in masonry, N-mm (lb-in).
M_n	=	Nominal moment strength, N-mm (lb-in).
M_s	=	Moment of tensile force in reinforcement about centroid of compressive force in masonry, N-mm (lb-in).
M_{ser}	=	Service moment at mid height of panel, including $P\Delta$ effects, N-mm (lb-in).
M_u	=	Factored moment, N-mm (lb-in).
N	=	Total vertical load, N (lb).
n	=	Modular ratio = E_s/E_m .
P	=	Design axial load, kN (lb).
P_a	=	Allowable centroidal axial load for reinforced masonry columns, kN (lb).
P_b	=	Nominal balanced design axial strength, kN (lb).
P_f	=	Load from tributary floor or roof area, kN (lb).
P_n	=	Nominal axial strength in masonry, kN (lb).

P_o	=	Nominal axial load strength in masonry without flexure, kN (lb).
P_u	=	Factored axial load, kN (lb).
P_{uf}	=	Factored load from tributary floor or roof loads, kN (lb).
P_{uw}	=	Factored weight of wall tributary to section under consideration, kN (lb).
P_w	=	Weight of wall tributary to section under consideration, kN (lb).
R	=	Numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral load resisting system.
R_w	=	Seismic response modification factor.
r	=	Radius of gyration (based on specified unit dimensions or Tables 9.8, 9.9 and 9.10 mm (in)).
r_b	=	Ratio of area of reinforcing bars cut off to total area of reinforcing bars at the section.
S	=	Section modulus, mm ³ (in ³).
s	=	Spacing of stirrups or of bent bars in direction parallel to that of main reinforcement, mm (in).
t	=	Effective thickness of wythe, wall or column, mm (in).
u	=	Bond stress per unit of surface area of reinforcing bar, MPa (psi).
V	=	Total design shear force, kN (lb).
V_{jh}	=	Total horizontal joint shear, kN (lb).
V_m	=	Nominal shear strength of masonry, kN (lb).
V_n	=	Nominal shear strength, kN (lb).
V_s	=	Nominal shear strength of shear reinforcement, kN (lb).
V_u	=	Required shear strength in masonry, kN (lb).
W	=	Wind load, or related internal moments and forces, kN (lb).
w_u	=	Factored distributed lateral load, kN (lb).
ΔM	=	Algebraic sum of moments at the end of structural member.
Δ_s	=	Horizontal deflection at mid height under factored load, mm (in).
Δ_u	=	Deflection due to factored loads, mm (in).
ρ	=	Ratio of area of flexural tensile reinforcement, A_s , to area bd .
ρ_b	=	Reinforcement ratio producing balanced strain conditions.
ρ_n	=	Ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv} .
Σ_o	=	Sum of perimeters of all longitudinal reinforcement, mm (in).
σ	=	Stress in masonry, N/mm ² (lb/in ²)
$\sqrt{f'_m}$	=	Square root of specified strength of masonry at the age of 28 days, MPa (psi).
ϕ	=	Strength-reduction factor.

9.2 Scope

The materials, design, construction and quality assurance of masonry shall be in accordance with this chapter.

9.2.1 Design Methods

Masonry shall comply with the provisions of one of the following design methods in this chapter.

9.2.1.1 Working stress design. Masonry designed by the working stress design method shall comply with the provisions of Sections 9.6 and 9.7.

9.2.1.2 Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 9.6 and 9.8.

9.2.1.3 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Section 9.9.

9.3 Definitions

Areas:

Bedded Area is the area of the surface of a masonry unit, which is in contact with mortar in the plane of the joint.

Effective Area of Reinforcement is the cross-sectional area of reinforcement multiplied by the cosine of the angle between the reinforcement and the direction for which effective area is to be determined.

Gross Area is the total cross-sectional area of a specified section.

Net Area is the gross cross-sectional area minus the area of ungrouted cores, notches, cells and unbedded areas. Net area is the actual surface area of a cross section of masonry.

Transformed Area is the equivalent area of one material to a second based on the ratio of moduli of elasticity of the first material to the second.

Bond:

Adhesion Bond is the adhesion between masonry units and mortar or grout.

Reinforcing Bond is the adhesion between steel reinforcement and mortar or grout.

Bond Beam is a horizontal grouted element within masonry in which reinforcement is embedded.

Cell is a void space having a gross cross-sectional area greater than 965 mm² (1.5 in²).

Cleanout is an opening to the bottom of a grout space of sufficient size and spacing to allow the removal of debris.

Collar Joint is the mortared or grouted space between wythes of masonry.

Column, Reinforced is a vertical structural member in which both the reinforcement and masonry resist compression.

Column, Unreinforced is a vertical structural member whose horizontal dimension measured at right angles to the thickness does not exceed three times the thickness.

Confining elements, the horizontal or vertical reinforced concrete members used in the confined masonry

Confined masonry, masonry provided with reinforced concrete confining elements in the vertical and horizontal directions.

Dimensions:

Actual Dimensions are the measured dimensions of a designated item. The actual dimension shall not vary from the specified dimension by more than the amount allowed in the appropriate standard of quality in Section 9.4.

Nominal Dimensions of masonry units are equal to its specified dimensions plus the thickness of the joint with which the unit is laid.

Specified Dimensions are the dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other component of a structure.

Grout Lift is an increment of grout height within the total grout pour.

Grout Pour is the total height of masonry wall to be grouted prior to the erection of additional masonry. A grout pour will consist of one or more grout lifts.

Grouted Masonry:

Grouted Hollow-unit Masonry is that form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

Grouted Multiwythe Masonry is that form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

Joints:

Bed Joint is the mortar joint that is horizontal at the time the masonry units are placed.

Head Joint is the mortar joint having a vertical transverse plane.

Masonry Units:

Masonry Unit is brick, tile, stone, glass block or concrete block conforming to the requirements specified in Section 9.4.

Hollow-Masonry Unit is a masonry unit whose net cross-sectional areas (solid area) in any plane parallel to the surface containing cores, cells or deep frogs is less than 75 percent of its gross cross-sectional area measured in the same plane.

Solid-Masonry Unit is a masonry unit whose net cross-sectional area in any plane parallel to the surface containing the cores or cells is at least 75 percent of the gross cross-sectional area measured in the same plane.

Prism is an assemblage of masonry units and mortar with or without grout used as a test specimen for determining properties of the masonry.

Reinforced Masonry is that form of masonry construction in which reinforcement acting in conjunction with the masonry is used to resist forces.

Shell is the outer portion of a hollow masonry unit as placed in masonry.

Simple masonry buildings, buildings complying with the requirements of Section 9.9.1.

Walls:

Bonded Wall is a masonry wall in which two or more wythes are bonded to act as a structural unit.

Cavity Wall is a wall containing continuous air space with a minimum width of 50 mm (2 in) and a maximum width of 115 mm (4.5 in) between wythes which are tied with metal ties.

Wall Tie is a mechanical metal fastener which connects wythes of masonry to each other or to other materials.

Web is an interior solid portion of a hollow-masonry unit as placed in masonry.

Wythe is the portion of a wall which is one masonry unit in thickness. A collar joint is not considered a wythe.

9.4 Material Standards

9.4.1 *Quality*

Materials used in masonry shall conform to the requirements stated herein. If no requirements are specified in this section for a material, quality shall be based on generally accepted good practice, subject to the approval of the building official. Reclaimed or previously used masonry units shall meet the applicable requirements as for new masonry units of the same material for their intended use.

9.4.2 *Standards of Quality*

1. *Aggregates*

- 1.1 ASTM C 144, Aggregates for Masonry Mortar
- 1.2 ASTM C 404, Aggregates for Grout

2. *Cement*

- 2.1 UBC Standard 21-11, Cement, Masonry. (Plastic cement conforming to the requirements of UBC Standard 25-1 may be used in lieu of masonry cement when it also conforms to UBC Standard 21-11.)
- 2.2 ASTM C 150, Portland Cement
- 2.3 UBC Standard 21-14, Mortar Cement

3. *Lime*

- 3.1 UBC Standard 21-12, Quicklime for Structural Purposes
- 3.2 UBC Standard 21-13, Hydrated Lime for Masonry Purposes. When Types N and NA hydrated lime are used in masonry mortar, they shall comply with the provisions of UBC Standard 21-15, Section 21.1506.7, excluding the plasticity requirement.

4. *Masonry units of clay or shale*

- 4.1 ASTM C 34, Structural Clay Load-bearing Wall Tile
- 4.2 ASTM C 56, Structural Clay Non-load-bearing Tile
- 4.3 UBC Standard 21-1, Section 21.101, Building Brick (solid units)
- 4.4 ASTM C 126, Ceramic Glazed Structural Clay Facing Tile, Facing Brick and Solid Masonry Units. Load bearing glazed brick shall conform to the weathering and structural requirements of UBC Standard 21-1, Section 21.106, Facing Brick
- 4.5 UBC Standard 21-1, Section 21.106, Facing Brick (solid units)

- 4.6 UBC Standard 21-1, Section 21.107, Hollow Brick
- 4.7 ASTM C 67, Sampling and Testing Brick and Structural Clay Tile
- 4.8 ASTM C 212, Structural Clay Facing Tile
- 4.9 ASTM C 530, Structural Clay Non-Load-bearing Screen Tile

5. *Masonry units of concrete*

- 5.1 UBC Standard 21-3, Concrete Building Brick
- 5.2 UBC Standard 21-4, Hollow and Solid Load-bearing Concrete Masonry Units
- 5.3 UBC Standard 21-5, Non-load-bearing Concrete Masonry Units
- 5.4 ASTM C 140, Sampling and Testing Concrete Masonry Units
- 5.5 ASTM C 426, Standard Test Method for Drying Shrinkage of Concrete Block

6. *Masonry units of other materials*

- 6.1 *Calcium silicate.* UBC Standard 21-2, Calcium Silicate Face Brick (Sand-lime Brick)
- 6.2 UBC Standard 21-9, Unburned Clay Masonry Units and Standard Methods of Sampling and Testing Unburned Clay Masonry Units
- 6.3 ACI-704, Cast Stone
- 6.4 UBC Standard 21-17, Test Method for Compressive Strength of Masonry Prisms

7. *Connectors*

- 7.1 Wall ties and anchors made from steel wire shall conform to UBC Standard 21-10, Part II, and other steel wall ties and anchors shall conform to A 36 in accordance with UBC Standard 22-1. Wall ties and anchors made from copper, brass or other nonferrous metal shall have minimum tensile yield strength of 207 MPa (30,000 psi).
- 7.2 All such items not fully embedded in mortar or grout shall either be corrosion resistant or shall be coated after fabrication with copper, zinc or a metal having at least equivalent corrosion-resistant properties.

8. *Mortar*

- 8.1 UBC Standard 21-15, Mortar for Unit Masonry and Reinforced Masonry other than Gypsum
- 8.2 UBC Standard 21-16, Field Tests Specimens for Mortar
- 8.3 UBC Standard 21-20, Standard Test Method for Flexural Bond Strength of Mortar Cement

9. *Grout*

- 9.1 UBC Standard 21-18, Method of Sampling and Testing Grout
- 9.2 UBC Standard 21-19, Grout for Masonry

10. *Reinforcement*

- 10.1 UBC Standard 21-10, Part I, Joint Reinforcement for Masonry
- 10.2 ASTM A 615, A 616, A 617, A 706, A 767, and A 775, Deformed and Plain Billet-steel Bars, Rail-steel Deformed and Plain Bars, Axle-steel

- Deformed and Plain Bars, and Deformed Low-alloy Bars for Concrete Reinforcement
- 10.3 UBC Standard 21-10, Part II, Cold-drawn Steel Wire for Concrete Reinforcement

9.4.3 *Mortar and Grout*

9.4.3.1 *General*

Mortar and grout shall comply with the provisions of this section. Special mortars, grouts or bonding systems may be used, subject to satisfactory evidence of their capabilities when approved by the building official.

9.4.3.2 *Materials*

Materials used as ingredients in mortar and grout shall conform to the applicable requirements in Section 9.4. Cementitious materials for grout shall be one or both of the following: lime and Portland cement. Cementitious materials for mortar shall be one or more of the following: lime, masonry cement, Portland cement and mortar cement. Cementitious materials or additives shall not contain epoxy resins and derivatives, phenols, asbestos fibers or fireclays. Water used in mortar or grout shall be clean and free of deleterious amounts of acid, alkalies or organic material or other harmful substances.

9.4.4 *Mortar*

9.4.4.1 *General.* Mortar shall consist of a mixture of cementitious materials and aggregate to which sufficient water and approved additives, if any, have been added to achieve a workable, plastic consistency.

9.4.4.2 *Selecting proportions.* Mortar with specified proportions of ingredients that differ from the mortar proportions of Table 9.1 may be approved for use when it is demonstrated by laboratory or field experience that this mortar with the specified proportions of ingredients, when combined with the masonry units to be used in the structure, will achieve the specified compressive strength f'_m . Water content shall be adjusted to provide proper workability under existing field conditions. When the proportion of ingredients is not specified, the proportions by mortar type shall be used as given in Table 9.1.

9.4.5 *Grout*

9.4.5.1 *General.* Grout shall consist of a mixture of cementitious materials and aggregate to which water has been added such that the mixture will flow without segregation of the constituents. The specified compressive strength of grout, f'_g , shall not be less than 14 MPa (2,000 psi).

9.4.5.2 *Selecting proportions.* Water content shall be adjusted to provide proper workability and to enable proper placement under existing field conditions, without segregation. Grout shall be specified by one of the following methods:

1. Proportions of ingredients and any additives shall be based on laboratory or field experience with the grout ingredients and the masonry units to be used. The grout shall be specified by the proportion of its constituents in terms of parts by volume.
2. Minimum compressive strength which will produce the required prism strength.
3. Proportions by grout type shall be used as given in Table 9.2.

9.4.6 Additives and Admixtures

9.4.6.1 General. Additives and admixtures to mortar or grout shall not be used unless approved by the building official.

9.4.6.2 Antifreeze compounds. Antifreeze liquids, chloride salts or other such substances shall not be used in mortar or grout.

9.4.6.3 Air entrainment. Air-entraining substances shall not be used in mortar or grout unless tests are conducted to determine compliance with the requirements of this code.

9.4.6.4 Colors. Only pure mineral oxide, carbon black or synthetic colors may be used. Carbon black shall be limited to a maximum of 3 percent of the weight of the cement.

9.4.7 Construction

9.4.7.1 General

Masonry shall be constructed according to the provisions of this section.

9.4.7.2 Materials: Handling, storage and preparation

All materials shall comply with applicable requirements of Section 9.4. Storage, handling and preparation at the site shall conform also to the following:

1. Masonry materials shall be stored so that at the time of use the materials are clean and structurally suitable for the intended use.
2. All metal reinforcement shall be free from loose rust and other coatings that would inhibit reinforcing bond.
3. At the time of laying, burned clay units and sand lime units shall have an initial rate of absorption not exceeding 1.6 liters/m² (0.035 ounce/in²) during a period of one minute. In the absorption test, the surface of the unit shall be held 3 mm (1/8 in) below the surface of the water.
4. Concrete masonry units shall not be wetted unless otherwise approved.
5. Materials shall be stored in a manner such that deterioration or intrusion of foreign materials is prevented and that the material will be capable of meeting applicable requirements at the time of mixing or placement.
6. The method of measuring materials for mortar and grout shall be such that proportions of the materials can be controlled.
7. Mortar or grout mixed at the jobsite shall be mixed for a period of time not less than three minutes or more than 10 minutes in a mechanical mixer with the amount of water required to provide the desired workability. Hand mixing of small amounts of mortar is permitted. Mortar may be re-tempered. Mortar or grout which has hardened or stiffened due to hydration of the cement shall not be used. In no case shall mortar be used two and one-half hours, nor grout used one and one-half hours, after the initial mixing water has been added to the dry ingredients at the jobsite.

Exception: Dry mixes for mortar and grout which are blended in the factory and mixed at the jobsite shall be mixed in mechanical mixers until workable, but not to exceed 10 minute.

9.4.8 Cold-weather Construction

9.4.8.1 General. All materials shall be delivered in a usable condition and stored to prevent wetting by capillary action, rain and snow. The tops of all walls not enclosed or sheltered shall be

covered with a strong weather-resistive material at the end of each day or shutdown. Partially completed walls shall be covered at all times when work is not in progress. Covers shall be draped over the wall and extend a minimum of 600 mm (2 ft) down both sides and shall be securely held in place, except when additional protection is required in Section 9.4.8.4.

9.4.8.2 Preparation. If ice or snow has inadvertently formed on a masonry bed, it shall be thawed by application of heat carefully applied until top surface of the masonry is dry to the touch. A section of masonry deemed frozen and damaged shall be removed before continuing construction of that section.

9.4.8.3 Construction. Masonry units shall be dry at time of placement. Wet or frozen masonry units shall not be laid. Special requirements for various temperature ranges are as follows:

1. Air temperature 4.5°C to 0°C (40°F to 32°F): Sand or mixing water shall be heated to produce mortar temperatures between 4.5°C and 49°C (40°F and 120°F).
2. Air temperature 0°C to -4°C (32°F to 25°F): Sand and mixing water shall be heated to produce mortar temperatures between 4.5°C and 49°C (40°F and 120°F). Maintain temperatures of mortar on boards above freezing.
3. Air temperature -4°C to -7°C (25°F to 20°F): Sand and mixing water shall be heated to produce mortar temperatures between 4.5°C and 49°C (40°F and 120°F). Maintain mortar temperatures on boards above freezing. Salamanders or other sources of heat shall be used on both sides of walls under construction. Windbreaks shall be employed when wind is in excess of 25 km/h (15 miles per hour).
4. Air temperature -7°C (20°F) and below: Sand and mixing water shall be heated to produce mortar temperatures between 4.5°C and 49°C (40°F and 120°F). Enclosure and auxiliary heat shall be provided to maintain air temperature above freezing. Temperature of units when laid shall not be less than -7°C (20°F).

9.4.8.4 Protection. When the mean daily air temperature is 4.5°C to 0°C (40°F to 32°F), masonry shall be protected from rain or snow for 24 hours by covering with a weather-resistive membrane. When the mean daily air temperature is 0°C to -4°C (32°F to 25°F), masonry shall be completely covered with a weather resistive membrane for 24 hours. When the mean daily air temperature is -4°C to -7°C (25°F to 20°F), masonry shall be completely covered with insulating blankets or equally protected for 24 hours. When the mean daily air temperature is -7°C (20°F) or below, masonry temperature shall be maintained above freezing for 24 hours by enclosure and supplementary heat, by electric heating blankets, infrared heat lamps or other approved methods.

9.4.8.5 Placing grout and protection of grouted masonry. When air temperatures fall below 4.5°C (40°F), grout mixing water and aggregate shall be heated to produce grout temperatures between 4.5°C and 49°C (40°F and 120°F). Masonry to be grouted shall be maintained above freezing during grout placement and for at least 24 hours after placement. When atmospheric temperatures fall below -7°C (20°F), enclosures shall be provided around the masonry during grout placement and for at least 24 hours after placement.

9.4.9 Placing Masonry Units

9.4.9.1 Mortar. The mortar shall be sufficiently plastic and units shall be placed with sufficient pressure to extrude mortar from the joint and produce a tight joint. Deep furrowing which produces voids shall not be used. The initial bed joint thickness shall not be less than 6 mm (1/4 inch) or more than 25 mm (1 inch); subsequent bed joints shall not be less than 6 mm (1/4 inch) or more than 15 mm (5/8 inch) in thickness.

9.4.9.2 Surfaces. Surfaces to be in contact with mortar or grout shall be clean and free of deleterious materials.

9.4.9.3 *Solid masonry units.* Solid masonry units shall have full head and bed joints.

9.4.9.4 *Hollow-masonry units.* All head and bed joints shall be filled solidly with mortar for a distance in from the face of the unit not less than the thickness of the shell. Head joints of open-end units with beveled ends that are to be fully grouted need not be mortared. The beveled ends shall form a grout key which permits grout within 15 mm (5/8 inch) of the face of the unit. The units shall be tightly butted to prevent leakage of grout.

9.4.10 *Reinforcement Placing*

Reinforcement details shall conform to the requirements of this chapter. Metal reinforcement shall be located in accordance with the plans and specifications. Reinforcement shall be secured against displacement prior to grouting by wire positioners or other suitable devices at intervals not exceeding 200 bar diameters. Tolerances for the placement of reinforcement in walls and flexural elements shall be plus or minus 12 mm (1/2 inch) for d equal to 200 mm (8 inches) or less, 25 mm (1 inch) for d equal to 600 mm (24 inches) or less but greater than 200 mm (8 inches) and 32 mm (1.25 inches) for d greater than 600mm (24 inches). Tolerance for longitudinal location of reinforcement shall be 50 mm (2 inches).

9.4.11 *Grouted Masonry*

9.4.11.1 *General conditions.* Grouted masonry shall be constructed in such a manner that all elements of the masonry act together as a structural element. Prior to grouting, the grout space shall be clean so that all spaces to be filled with grout do not contain mortar projections greater than 12 mm (1/2 inch), mortar droppings or other foreign material. Grout shall be placed so that all spaces designated to be grouted shall be filled with grout and the grout shall be confined to those specific spaces.

Grout materials and water content shall be controlled to provide adequate fluidity for placement without segregation of the constituents, and shall be mixed thoroughly. The grouting of any section of wall shall be completed in one day with no interruptions greater than one hour.

Between grout pours, a horizontal construction joint shall be formed by stopping all wythes at the same elevation and with the grout stopping a minimum of 38 mm (1.5 inches) below a mortar joint, except at the top of the wall. Where bond beams occur, the grout pour shall be stopped a minimum of 12 mm (1/2 inch) below the top of the masonry.

Size and height limitations of the grout space or cell shall not be less than shown in Table 9.3. Higher grout pours or smaller cavity widths or cell size than shown in Table 9.3 may be used when approved, if it is demonstrated that grout spaces will be properly filled.

Cleanouts shall be provided for all grout pours over 1525 mm (5 ft) in height.

Where required, cleanouts shall be provided in the bottom course at every vertical bar but shall not be spaced more than 810 mm (32 inches) on center for solidly grouted masonry. When cleanouts are required, they shall be sealed after inspection and before grouting. Where cleanouts are not provided, special provisions must be made to keep the bottom and sides of the grout spaces, as well as the minimum total clear area as required by Table 9.3, clean and clear prior to grouting.

Units may be laid to the full height of the grout pour and grout shall be placed in a continuous pour in grout lifts not exceeding 1830 mm (6 feet). When approved, grout lifts may be greater than 1830 mm (6 feet) if it can be demonstrated the grout spaces can be properly filled.

All cells and spaces containing reinforcement shall be filled with grout.

9.4.11.2 Construction requirements. Reinforcement shall be placed prior to grouting. Bolts shall be accurately set with templates or by approved equivalent means and held in place to prevent dislocation during grouting.

Segregation of the grout materials and damage to the masonry shall be avoided during the grouting process.

Grout shall be consolidated by mechanical vibration during placement before loss of plasticity in a manner to fill the grout space. Grout pours greater than 300 mm (12 inches) in height shall be reconsolidated by mechanical vibration to minimize voids due to water loss. Grout pours 300 mm (12 inches) or less in height shall be mechanically vibrated or puddled.

In one-storey buildings having wood-frame exterior walls, foundations not over 600 mm (24 inches) high measured from the top of the footing may be constructed of hollow-masonry units laid in running bond without mortared head joints. Any standard shape unit may be used, provided the masonry units permit horizontal flow of grout to adjacent units. Grout shall be solidly poured to the full height in one lift and shall be puddled or mechanically vibrated.

In nonstructural elements which do not exceed 2.5 m (8 feet) in height above the highest point of lateral support, including fireplaces and residential chimneys, mortar of pouring consistency may be substituted for grout when the masonry is constructed and grouted in pours of 300 mm (12 inches) or less in height. In multiwythe grouted masonry, vertical barriers of masonry shall be built across the grout space the entire height of the grout pour and spaced not more than 9.15 m (30 feet) horizontally. The grouting of any section of wall between barriers shall be completed in one day with no interruption longer than one hour.

9.4.12 Aluminum Equipment

Grout shall not be handled nor pumped utilizing aluminum equipment unless it can be demonstrated with the materials and equipment to be used that there will be no deleterious effect on the strength of the grout.

9.4.13 Joint Reinforcement

Wire joint reinforcement used in the design as principal reinforcement in hollow-unit construction shall be continuous between supports unless splices are made by lapping:

1. Fifty-four wire diameters in a grouted cell, or
2. Seventy-five wire diameters in the mortared bed joint, or
3. In alternate bed joints of running bond masonry a distance not less than 54 diameters plus twice the spacing of the bed joints, or
4. As required by calculation and specific location in areas of minimum stress, such as points of inflection. Side wires shall be deformed and shall conform to UBC Standard 21-10, Part I, Joint Reinforcement for Masonry.

9.5 Quality Assurance

9.5.1 General

Quality assurance shall be provided to ensure that materials, construction and workmanship are in compliance with the plans and specifications, and the applicable requirements of this chapter. When required, inspection records shall be maintained and made available to the building official.

9.5.2 Scope

Quality assurance shall include, but is not limited to, assurance that:

1. Masonry units, reinforcement, cement, lime, aggregate and all other materials meet the requirements of the applicable standards of quality and that they are properly stored and prepared for use.
2. Mortar and grout are properly mixed using specified proportions of ingredients. The method of measuring materials for mortar and grout shall be such that proportions of materials are controlled.
3. Construction details, procedures and workmanship are in accordance with the plans and specifications.
4. Placement, splices and reinforcement sizes are in accordance with the provisions of this chapter and the plans and specifications.

9.5.3 Compliance with f'_{m}

9.5.3.1 General. Compliance with the requirements for the specified compressive strength of masonry f'_{m} shall be in accordance with one of the sections in this subsection.

9.5.3.2 Masonry prism testing. The compressive strength of masonry determined in accordance with UBC Standard 21-17 for each set of prisms shall equal or exceed f'_{m} . Compressive strength of prisms shall be based on tests at 28 days. Compressive strength at seven days or three days may be used provided a relationship between seven-day and three-day and 28-day strength has been established for the project prior to the start of construction. Verification by masonry prism testing shall meet the following:

1. A set of five masonry prisms shall be built and tested in accordance with UBC Standard 21-17 prior to the start of construction. Materials used for the construction of the prisms shall be taken from those specified to be used in the project. Prisms shall be constructed under the observation of the engineer or special inspector or an approved agency and tested by an approved agency.
2. When full allowable stresses are used in design, a set of three prisms shall be built and tested during construction in accordance with UBC Standard 21-17 for each 465 m^2 ($5,000 \text{ ft}^2$) of wall area, but not less than one set of three masonry prisms for the project.
3. When one half the allowable masonry stresses are used in design, testing during construction is not required. A letter of certification from the supplier of the materials used to verify the f'_{m} in accordance with Section 9.5.3.2, Item 1, shall be provided at the time of, or prior to, delivery of the materials to the jobsite to ensure the materials used in construction are representative of the materials used to construct the prisms prior to construction.

9.5.3.3 Masonry prism test record. Compressive strength verification by masonry prism test records shall meet the following:

1. A masonry prism test record approved by the building official of at least 30 masonry prisms which were built and tested in accordance with UBC Standard 21-17. Prisms shall have been constructed under the observation of an engineer or special inspector or an approved agency and shall have been tested by an approved agency.
2. Masonry prisms shall be representative of the corresponding construction.
3. The average compressive strength of the test record shall equal or exceed $1.33 f'_{m}$.
4. When full allowable stresses are used in design, a set of three masonry prisms shall be built during construction in accordance with UBC Standard 21-17 for each 465 m^2 ($5,000 \text{ ft}^2$) of wall area, but not less than one set of three prisms for the project.

5. When one half the allowable masonry stresses are used in design, field testing during construction is not required. A letter of certification from the supplier of the materials to the jobsite shall be provided at the time of, or prior to, delivery of the materials to assure the materials used in construction are representative of the materials used to develop the prism test record in accordance with Section 9.5.3.3, Item 1.

9.5.3.4 Unit strength method. Verification by the unit strength method shall meet the following:

1. When full allowable stresses are used in design, units shall be tested prior to construction and test units during construction for each 465 m^2 ($5,000 \text{ ft}^2$) of wall area for compressive strength to show compliance with the compressive strength required in Table 9.4; and

Exception: Prior to the start of construction, prism testing may be used in lieu of testing the unit strength. During construction, prism testing may also be used in lieu of testing the unit strength and the grout as required by Section 9.5.3.4, Item 4.

2. When one half the allowable masonry stresses are used in design, testing is not required for the units. A letter of certification from the manufacturer of the units shall be provided at the time of, or prior to, delivery of the units to the jobsite to assure the units comply with the compressive strength required in Table 9.4; and
3. Mortar shall comply with the mortar type required in Table 9.4; and
4. When full stresses are used in design for concrete masonry, grout shall be tested for each 465 m^2 ($5,000 \text{ ft}^2$) of wall area, but not less than one test per project, to show compliance with the compressive strength required in Table 9.4, Footnote 4.
5. When one half the allowable stresses are used in design for concrete masonry, testing is not required for the grout. A letter of certification from the supplier of the grout shall be provided at the time of, or prior to, delivery of the grout to the jobsite to assure the grout complies with the compressive strength required in Table 9.4, Footnote 4; or
6. When full allowable stresses are used in design for clay masonry, grout proportions shall be verified by the engineer or special inspector or an approved agency to conform with Table 9.2.
7. When one half the allowable masonry stresses are used in design for clay masonry, a letter of certification from the supplier of the grout shall be provided at the time of, or prior to, delivery of the grout to the jobsite to assure the grout conforms to the proportions of Table 9.2.

9.5.3.5 Testing prisms from constructed masonry. When approved by the building official, acceptance of masonry which does not meet the requirements of Section 9.5.3.2, 9.5.3.3 or 9.5.3.4 shall be permitted to be based on tests of prisms cut from the masonry construction in accordance with the following:

1. A set of three masonry prisms that are at least 28 days old shall be saw cut from the masonry for each 465 m^2 ($5,000 \text{ ft}^2$) of the wall area that is in question but not less than one set of three masonry prisms for the project. The length, width and height dimensions of the prisms shall comply with the requirements of UBC Standard 21-17. Transporting, preparation and testing of prisms shall be in accordance with UBC Standard 21-17.
2. The compressive strength of prisms shall be the value calculated in accordance with UBC Standard 21-17, Section 21.1707.2, except that the net cross-sectional area of the prism shall be based on the net mortar bedded area.
3. Compliance with the requirement for the specified compressive strength of masonry, f'_m , shall be considered satisfied provided the modified compressive strength equals or exceeds the specified f'_m . Additional testing of specimens cut from locations in question shall be permitted.

9.5.4 *Mortar Testing*

When required, mortar shall be tested in accordance with UBC Standard 21-16.

9.5.5 *Grout Testing*

When required, grout shall be tested in accordance with UBC Standard 21-18.

9.6 General Design Requirements

9.6.1 *General*

9.6.1.1 Scope. The design of masonry structures shall comply with the working stress design provisions of Section 9.7, or the strength design provisions of Section 9.8 or the empirical design provisions of Section 9.9, and with the provisions of this section. Unless otherwise stated, all calculations shall be made using or based on specified dimensions.

9.6.1.2 Plans. Plans submitted for approval shall describe the required design strengths of masonry materials and inspection requirements for which all parts of the structure were designed, and any load test requirements.

9.6.1.3 Design loads. See Chapter 5 for design loads

9.6.1.4 Stack bond. In bearing and nonbearing walls, except veneer walls, if less than 75 percent of the units in any transverse vertical plane lap the ends of the units below a distance less than one half the height of the unit, or less than one fourth the length of the unit, the wall shall be considered laid in stack bond.

9.6.1.5 *Multiwythe walls.*

9.6.1.5.1 General. All wythes of multiwythe walls shall be bonded by grout or tied together by corrosion-resistant wall ties of joint reinforcement conforming to the requirements of Section 9.4, and as set forth in this section.

9.6.1.5.2 Wall ties in cavity wall construction. Wall ties shall be of sufficient length to engage all wythes. The portion of the wall ties within the wythe shall be completely embedded in mortar or grout. The ends of the wall ties shall be bent to 90-degree angles with an extension not less than 50 mm (2 inches) long. Wall ties not completely embedded in mortar or grout between wythes shall be a single piece with each end engaged in each wythe.

There shall be at least one 10 mm (3/8 inch) diameter wall tie for each 0.40 m^2 (4.5 ft^2) of wall area. For cavity walls in which the width of the cavity is greater than 75 mm (3 in), but not more than 115 mm (4.5 in), at least one 10 mm (3/8 inch) diameter wall tie for each 0.30 m^2 (3 ft^2) of wall area shall be provided. Ties in alternate courses shall be staggered.

The maximum vertical distance between ties shall not exceed 600 mm (24 inches) and the maximum horizontal distance between ties shall not exceed 914 mm (36 inches). Additional ties spaced not more than 910 mm (36 inches) apart shall be provided around openings within a distance of 300 mm (12 inches) from the edge of the opening. Adjustable wall ties shall meet the following requirements:

1. One tie shall be provided for each 0.15 m^2 (1.75 ft^2) of wall area. Horizontal and vertical spacing shall not exceed 400 mm (16 inches). Maximum misalignment of bed joints from one wythe to the other shall be 32 mm (1.25 inches).

2. Maximum clearance between the connecting parts of the tie shall be 1.5 mm (1/16 inch). When used, pintle ties shall have at least two 5 mm (3/16 inch) diameter pintle legs. Wall ties of different size and spacing that provide equivalent strength between wythes may be used.

9.6.1.5.3 *Wall ties for grouted multiwythe construction.* Wythes of multiwythe walls shall be bonded together with at least 5 mm (3/16 inch) steel wall tie for each 0.2 m² (2 ft²) of area. Wall ties of different size and spacing that provide equivalent strength between wythes may be used.

9.6.1.5.4 *Joint reinforcement.* Prefabricated joint reinforcement for masonry walls shall have at least one cross wire of at least No. 9 gage steel for each 0.2 m² (2 ft²) of wall area. The vertical spacing of the joint reinforcement shall not exceed 400 mm (16 inches). The longitudinal wires shall be thoroughly embedded in the bed joint mortar. The joint reinforcement shall engage all wythes. Where the space between tied wythes is solidly filled with grout or mortar, the allowable stresses and other provisions for masonry bonded walls shall apply. Where the space is not filled, tied walls shall conform to the allowable stress, lateral support, thickness (excluding cavity), height and tie requirements for cavity walls.

9.6.1.6 *Vertical support.* Structural members providing vertical support of masonry shall provide a bearing surface on which the initial bed joint shall not be less than 6 mm (1/4 inch) or more than 25 mm (1 inch) in thickness and shall be of noncombustible material, except where masonry is a nonstructural decorative feature or wearing surface.

9.6.1.7 *Lateral support.* Lateral support of masonry may be provided by cross walls, columns, pilasters, counterforts or buttresses where spanning horizontally or by floors, beams, grits or roofs where spanning vertically. The clear distance between lateral supports of a beam shall not exceed 32 times the least width of the compression area.

9.6.1.8 *Protection of ties and joint reinforcement.* A minimum of 16 mm (5/8 inch) mortar cover shall be provided between ties or joint reinforcement and any exposed face. The thickness of grout or mortar between masonry units and joint reinforcement shall not be less than 6 mm (1/4 inch) except that 6 mm (1/4 inch) or smaller diameter reinforcement or bolts may be placed in bed joints which are at least twice the thickness of the reinforcement or bolts.

9.6.1.9 *Pipes and conduits embedded in masonry.* Pipes or conduit shall not be embedded in any masonry in a manner that will reduce the capacity of the masonry to less than that necessary for required strength or required fire protection. Placement of pipes or conduits in unfilled cores of hollow-unit masonry shall not be considered as embedment.

Exceptions:

1. Rigid electric conduits may be embedded in structural masonry when their locations have been detailed on the approved plan.
2. Any pipe or conduit may pass vertically or horizontally through any masonry by means of a sleeve at least large enough to pass any hub or coupling on the pipeline. Such sleeves shall not be placed closer than three diameters, center to center, nor shall they unduly impair the strength of construction.

9.6.1.10 *Load tests.* When a load test is required, the member or portion of the structure under consideration shall be subjected to a superimposed load equal to twice the design live load plus one half of the dead load. This load shall be left in position for a period of 24 hours before removal. If, during the test or upon removal of the load, the member or portion of the structure shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or where approved, a lower rating shall be established.

A flexural member shall be considered to have passed the test if the maximum deflection D at the end of the 24-hour period does not exceed the value of Formulas (9.6-1) or (9.6-2) and the beams and slabs show a recovery of at least 75 percent of the observed deflection within 24 hours after removal of the load.

$$D = \frac{l}{200} \quad (9.6-1)$$

$$D = \frac{l^2}{4000t} \quad (9.6-2)$$

9.6.1.11 Reuse of masonry units. Masonry units may be reused when clean, whole and conforming to the other requirements of this section. All structural properties of masonry of reclaimed units shall be determined by approved test.

9.6.1.12 Special provisions in areas of seismic risk

9.6.1.12.1 General. Masonry structures constructed in the seismic zones shown in Fig. 2.1 shall be designed in accordance with the design requirements of this chapter and the special provisions for each seismic zone given in this section.

9.6.1.12.2 Special provisions for Seismic Zone 1. There are no special design and construction provisions in this section for structures built in Seismic Zone 1.

9.6.1.12.3 Special provisions for Seismic Zone 2. Masonry structures in Seismic Zone 2 shall comply with the following special provisions:

1. Columns shall be reinforced as specified in Sections 9.6.3.6, 9.6.3.7 and 9.7.2.13.
2. Vertical wall reinforcement of at least 130 mm^2 (0.20 in^2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening, at the ends of walls and at maximum spacing of 1220 mm (4 ft) apart horizontally throughout walls.
3. Horizontal wall reinforcement not less than 130 mm^2 (0.20 in^2) in cross-sectional area shall be provided (1) at the bottom and top of wall openings and shall extend not less than 600 mm (24 inches) or less than 40 bar diameters past the opening, (2) continuously at structurally connected roof and floor levels and at the top of walls, (3) at the bottom of walls or in the top of foundations when doweled in walls, and (4) at maximum spacing of 3050 mm (10 feet) unless uniformly distributed joint reinforcement is provided. Reinforcement at the top and bottom of openings when continuous in walls may be used in determining the maximum spacing specified in Item 1 of this paragraph.
4. Where stack bond is used, the minimum horizontal reinforcement ratio shall be $0.0007bt$. This ratio shall be satisfied by uniformly distributed joint reinforcement or by horizontal reinforcement spaced not over 1220 mm (4 ft) and fully embedded in grout or mortar.
5. The following materials shall not be used as part of the vertical or lateral load-resisting systems: Type O mortar, masonry cement, plastic cement, non-load-bearing masonry units and glass block.

9.6.1.12.4 Special provisions for Seismic Zones 3 and 4. All masonry structures built in Seismic Zones 3 and 4 shall be designed and constructed in accordance with requirements for Seismic Zone 2 and with the following additional requirements and limitations:

Exception: One and two-storey masonry buildings of Group R, Division 3 and Group U Occupancies located in Seismic Zone 3 having masonry wall h/t ratios not greater than 27 and using running bond construction when provisions of Section 9.6.1.12.3 are met.

1. *Column reinforcement ties*

In columns that are stressed by tensile or compressive axial overturning forces from seismic loading, the spacing of column ties shall not exceed 200 mm (8 inches) for the full height of such columns. In all other columns, ties shall be spaced a maximum of 200 mm (8 inches) in the tops and bottoms of the columns for a distance of one sixth of the clear column height, 450 mm (18 inches) or the maximum column cross-sectional dimension, whichever is greater.

Tie spacing for the remaining column height shall not exceed the lesser of 16 bar diameters, 48 tie diameters, the least column cross-sectional dimension, or 450 mm (18 inches).

Column ties shall terminate with a minimum 135-degree hook with extensions not less than six bar diameters or 100 mm (4 inches). Such extensions shall engage the longitudinal column reinforcement and project into the interior of the column. Hooks shall comply with Section 9.7.2.2.5, Item 3.

Exception: Where ties are placed in horizontal bed joints, hooks shall consist of a 90-degree bend having an inside radius of not less than four tie diameters plus an extension of 32 tie diameters.

2. *Shear walls*

2.1 *Reinforcement*

The portion of the reinforcement required to resist shear shall be uniformly distributed and shall be joint reinforcement, deformed bars or a combination thereof. The spacing of reinforcement in each direction shall not exceed one half the length of the element, nor one half the heights of the element, nor 1200 mm (48 inches). Joint reinforcement used in exterior walls and considered in the determination of the shear strength of the member shall be hot-dipped galvanized in accordance with UBC Standard 21-10.

Reinforcement required to resist in-plane shear shall be terminated with a standard hook as defined in Section 9.7.2.2.5 or with an extension of proper embedment length beyond the reinforcement at the end of the wall section. The hook or extension may be turned up, down or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

2.2 *Bond*

Multiwythe grouted masonry shear walls shall be designed with consideration of the adhesion bond strength between the grout and masonry units. When bond strengths are not known from previous tests, the bond strength shall be determined by tests.

2.3 *Wall reinforcement*

All walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall, and the minimum area of reinforcement in either direction shall not be less than 0.0007 times the gross cross-sectional area of the wall. The minimum steel requirements for Seismic Zone 2 in Section 9.6.1.12.3, Items 2 and 3, may be included in the sum. The spacing of reinforcement shall not exceed 1200 mm (4 feet). The diameter of reinforcement shall not be less than 10 mm (3/8 inch) except that joint reinforcement may be considered as a part or the entire requirement for minimum reinforcement. Reinforcement shall be continuous around wall corners and through intersections. Only reinforcement which is continuous in the wall or element

shall be considered in computing the minimum area of reinforcement. Reinforcement with splices conforming to Section 9.7.2.2.6 shall be considered as continuous reinforcement.

2.4 *Stack bond*

Where stack bond is used, the minimum horizontal reinforcement ratio shall be $0.0015bt$. Where open-end units are used and grouted solid, the minimum horizontal reinforcement ratio shall be $0.0007bt$.

Reinforced hollow-unit stacked bond construction which is part of the seismic-resisting system shall use open-end units so that all head joints are made solid, shall use bond beam units to facilitate the flow of grout and shall be grouted solid.

3. *Type N mortar.*

Type N mortar shall not be used as part of the vertical- or lateral-load-resisting system.

4. *Concrete abutting structural masonry*

Concrete abutting structural masonry, such as at starter courses or at wall intersections not designed as true separation joints, shall be roughened to a full amplitude of 1.6 mm (1/16 inch) and shall be bonded to the masonry in accordance with the requirements of this chapter as if it were masonry. Unless keys or proper reinforcement is provided, vertical joints as specified in Section 9.6.1.4 shall be considered to be stack bond and the reinforcement as required for stack bond shall extend through the joint and be anchored into the concrete.

9.6.2 *Working Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry.*

9.6.2.1 *General.* In addition to the requirements of Section 9.6.1, the design of masonry structures by the working stress design method and strength design method shall comply with the requirements of this section. Additionally, the design of reinforced masonry structures by these design methods shall comply with the requirements of Section 9.6.3.

9.6.2.2 *Specified compressive strength of masonry.* The allowable stresses for the design of masonry shall be based on a value of f'_m selected for the construction. Verification of the value of f'_m shall be based on compliance with Section 9.5.3. Unless otherwise specified, f'_m shall be based on 28-day tests. If other than a 28-day test age is used, the value of f'_m shall be as indicated in design drawings or specifications. Design drawings shall show the value of f'_m for which each part of the structure is designed.

9.6.2.3 *Effective thickness.*

9.6.2.3.1 *Single-wythe walls.* The effective thickness of single-wythe walls of either solid or hollow units is the specified thickness of the wall.

9.6.2.3.2 *Multiwythe walls.* The effective thickness of multiwythe walls is the specified thickness of the wall if the space between wythes is filled with mortar or grout. For walls with an open space between wythes, the effective thickness shall be determined as for cavity walls.

9.6.2.3.3 *Cavity walls.* Where both wythes of a cavity wall are axially loaded, each wythe shall be considered to act independently and the effective thickness of each wythe is as defined in Section 9.6.2.3.1. Where only one wythe is axially loaded, the effective thickness of the cavity wall is taken as the square root of the sum of the squares of the specified thicknesses of the wythes.

Where a cavity wall is composed of a single wythe and a multiwythe, and both sides are axially loaded, each side of the cavity wall shall be considered to act independently and the effective thickness of each side is as defined in Sections 9.6.2.3.1 and 9.6.2.3.2. Where only one side is axially loaded, the effective thickness of the cavity wall is the square root of the sum of the squares of the specified thicknesses of the sides.

9.6.2.3.4 Columns. The effective thickness for rectangular columns in the direction considered is the specified thickness. The effective thickness for nonrectangular columns is the thickness of the square column with the same moment of inertia about its axis as that about the axis considered in the actual column.

9.6.2.4 Effective height. The effective height of columns and walls shall be taken as the clear height of members laterally supported at the top and bottom in a direction normal to the member axis considered. For members not supported at the top normal to the axis considered, the effective height is twice the height of the member above the support. Effective height less than clear height may be used if justified.

9.6.2.5 Effective area. The effective cross-sectional area shall be based on the minimum bedded area of hollow units, or the gross area of solid units plus any grouted area. Where hollow units are used with cells perpendicular to the direction of stress, the effective area shall be the lesser of the minimum bedded area or the minimum cross-sectional area. Where bed joints are raked, the effective area shall be correspondingly reduced. Effective areas for cavity walls shall be that of the loaded wythes.

9.6.2.6 Effective width of intersecting walls. Where a shear wall is anchored to an intersecting wall or walls, the width of the overhanging flange formed by the intersected wall on either side of the shear wall, which may be assumed working with the shear wall for purposes of flexural stiffness calculations, shall not exceed six times the thickness of the intersected wall. Limits of the effective flange may be waived if justified. Only the effective area of the wall parallel to the shear forces may be assumed to carry horizontal shear.

9.6.2.7 Distribution of concentrated vertical loads in walls. The length of wall laid up in running bond which may be considered capable of working at the maximum allowable compressive stress to resist vertical concentrated loads shall not exceed the center-to-center distance between such loads, nor the width of bearing area plus four times the wall thickness. Concentrated vertical loads shall not be assumed to be distributed across continuous vertical mortar or control joints unless elements designed to distribute the concentrated vertical loads are employed.

9.6.2.8 Loads on nonbearing walls. Masonry walls used as interior partitions or as exterior surfaces of a building which do not carry vertical loads imposed by other elements of the building shall be designed to carry their own weight plus any superimposed finish and lateral forces. Bonding or anchorage of nonbearing walls shall be adequate to support the walls and to transfer lateral forces to the supporting elements.

9.6.2.9 Vertical deflection. Elements supporting masonry shall be designed so that their vertical deflection will not exceed 1/600 of the clear span under total loads. Lintels shall bear on supporting masonry on each end such that allowable stresses in the supporting masonry are not exceeded. A minimum bearing length of 100 mm (4 inches) shall be provided for lintels bearing on masonry.

9.6.2.10 Structural continuity. Intersecting structural elements intended to act as a unit shall be anchored together to resist the design forces.

9.6.2.11 Walls intersecting with floors and roofs. Walls shall be anchored to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorage to such walls shall be designed to resist the horizontal force.

9.6.2.12 Modulus of elasticity of material

9.6.2.12.1 Modulus of elasticity of masonry. The moduli for masonry may be estimated as provided below. Actual values, where required, shall be established by test. The modulus of elasticity of masonry shall be determined by the secant method in which the slope of the line for the modulus of elasticity is taken from $0.05 f'_m$ to a point on the curve at $0.33 f'_m$. These values are not to be reduced by one half as set forth in Section 9.7.1.2.

Modulus of elasticity of clay or shale unit masonry.

$$E_m = 750 f'_m , 20.5 \text{GPa} \text{ (3,000,000 psi) maximum} \quad (9.6-3)$$

Modulus of elasticity of concrete unit masonry.

$$E_m = 750 f'_m , 20.5 \text{GPa} \text{ (3,000,000 psi) maximum} \quad (9.6-4)$$

9.6.2.12.2 Modulus of elasticity of steel

$$E_s = 200 \text{ GPa} \text{ (29,000,000 psi)} \quad (9.6-5)$$

9.6.2.13 Shear modulus of masonry

$$G = 0.4 E_m \quad (9.6-6)$$

9.6.2.14 Placement of embedded anchor bolts

9.6.2.14.1 General. Placement requirements for plate anchor bolts, headed anchor bolts and bent bar anchor bolts shall be determined in accordance with this subsection. Bent bar anchor bolts shall have a hook with a 90-degree bend with an inside diameter of three bolt diameters, plus an extension of one and one half bolt diameters at the free end.

Plate anchor bolts shall have a plate welded to the shank to provide anchorage equivalent to headed anchor bolts. The effective embedment depth 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 anchorage, and l_b for bent bar anchors shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter. All bolts shall be grouted in place with at least 25 mm (1 inch) of grout between the bolt and the masonry, except that 6 mm (1/4 inch) diameter bolts may be placed in bed joints which are at least 12 mm (1/2 inch) in thickness.

9.6.2.14.2 Minimum edge distance. The minimum anchor bolt edge distance l_{be} measured from the edge of the masonry parallel with the anchor bolt to the surface of the anchor bolt shall be 38 mm (1.5 inches).

9.6.2.14.3 Minimum embedment depth. The minimum embedment depth of anchor bolts l_b shall be four bolt diameters but not less than 50 mm (2 inches).

9.6.2.14.4 Minimum spacing between bolts. The minimum center-to-center distance between anchor bolts shall be four bolt diameters.

9.6.2.15 Flexural resistance of cavity walls. For computing the flexural resistance of cavity walls, lateral loads perpendicular to the plane of the wall shall be distributed to the wythes according to their respective flexural rigidities.

9.6.3 Working Stress Design and Strength Design Requirements for Reinforced Masonry

9.6.3.1 General. In addition to the requirements of Sections 9.6.1 and 9.6.2, the design of reinforced masonry structures by the working stress design method or the strength design method shall comply with the requirements of this section.

9.6.3.2 Plain bars. The use of plain bars larger than 6 mm (1/4 inch) in diameter is not permitted.

9.6.3.3 Spacing of longitudinal reinforcement. The clear distance between parallel bars, except in columns, shall not be less than the nominal diameter of the bars or 25 mm (1 inch), except that bars in a splice may be in contact. This clear distance requirement applies to the clear distance between a contact splice and adjacent splices or bars. The clear distance between the surface of a bar and any surface of a masonry unit shall not be less than 6 mm (1/4 inch) for fine grout and 12 mm (1/2 inch) for coarse grout. Cross webs of hollow units may be used as support for horizontal reinforcement.

9.6.3.4 Anchorage of flexural reinforcement. The tension or compression in any bar at any section shall be developed on each side of that section by the required development length. The development length of the bar may be achieved by a combination of an embedment length, anchorage or, for tension only, hooks.

Except at supports or at the free end of cantilevers, every reinforcing bar shall be extended beyond the point at which it is no longer needed to resist tensile stress for a distance equal to 12 bar diameters or the depth of the beam, whichever is greater. No flexural bar shall be terminated in a tensile zone unless at least one of the following conditions is satisfied:

1. The shear is not over one half that permitted, including allowance for shear reinforcement where provided.
2. Additional shear reinforcement in excess of that required is provided each way from the cutoff a distance equal to the depth of the beam. The shear reinforcement spacing shall not exceed $d/8r_b$.
3. The continuing bars provide double the area required for flexure at that point or double the perimeter required for reinforcing bond.

At least one third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance sufficient to develop one half the allowable stresses in the bar, not less than 1/16 of the clear span, or the depth d of the member, whichever is greater.

Tensile reinforcement for negative moment in any span of a continuous restrained or cantilever beam, or in any member of a rigid frame, shall be adequately anchored by reinforcement bond, hooks or mechanical anchors in or through the supporting member. At least one third of the required positive moment reinforcement in simple beams or at the freely supported end of continuous beams shall extend along the same face of the beam into the support at least 150 mm (6 inches).

At least one fourth of the required positive moment reinforcement at the continuous end of continuous beams shall extend along the same face of the beam into the support at least 150 mm (6 inches). Compression reinforcement in flexural members shall be anchored by ties or stirrups not less than 6 mm (1/4 inch) in diameter, spaced not farther apart than 16 bar diameters or 48 tie diameters, whichever is less. Such ties or stirrups shall be used throughout the distance where compression reinforcement is required.

9.6.3.5 Anchorage of shear reinforcement. Single, separate bars used as shear reinforcement shall be anchored at each end by one of the following methods:

1. Hooking tightly around the longitudinal reinforcement through 180 degrees.
2. Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress in the bar for plain or deformed bars.
3. By a standard hook, as defined in Section 9.7.2.2.5, considered as developing 50 MPa (7,500 psi), plus embedment sufficient to develop the remainder of the stress to which the bar is subjected. The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

The ends of bars forming a single U or multiple U stirrup shall be anchored by one of the methods set forth in Items 1 through 3 above or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 stirrup diameters. The loops or closed ends of simple U or multiple U stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees and project beyond the end of the bend at least 12 stirrup diameters.

9.6.3.6 Lateral ties. All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars by the corner of a complete tie having an included angle of not more than 135 degrees or by a standard hook at the end of a tie. The corner bars shall have such support provided by a complete tie enclosing the longitudinal bars. Alternate longitudinal bars shall have such lateral support provided by ties and no bar shall be farther than 150 mm (6 inches) from such laterally supported bar.

Lateral ties and longitudinal bars shall be placed not less than 40 mm (1.5 inches) and not more than 127 mm (5 inches) from the surface of the column. Lateral ties may be placed against the longitudinal bars or placed in the horizontal bed joints where the requirements of Section 9.6.1.8 are met. Spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie diameters or the least dimension of the column but not more than 450 mm (18 inches).

Ties shall be at least 6 mm (1/4 inch) in diameter for 22mm diameter (#7) or smaller longitudinal bars and at least 10 mm diameter (#3) for longitudinal bars larger than 22 mm diameter (No. 7). Ties smaller than 10 mm diameter (#3) may be used for longitudinal bars larger than 22 mm diameter (#7), provided the total cross-sectional area of such smaller ties crossing a longitudinal plane is equal to that of the larger ties at their required spacing.

9.6.3.7 Column anchor bolt ties. Additional ties shall be provided around anchor bolts which are set in the top of columns. Such ties shall engage at least four bolts or, alternately, at least four vertical column bars or a combination of bolts and bars totaling at least four. Such ties shall be located within the top 125 mm (5 inches) of the column and shall provide a total of 260mm² (0.4 in²) or more in cross-sectional area. The uppermost tie shall be within 50 mm (2 inches) of the top of the column.

9.6.3.8 Effective width b of compression area. In computing flexural stresses in walls where reinforcement occurs, the effective width assumed for running bond masonry shall not exceed six times the nominal wall thickness or the center-to-center distance between reinforcement. Where

stack bond is used, the effective width shall not exceed three times the nominal wall thickness or the center-to-center distance between reinforcement or the length of one unit, unless solid grouted open-end units are used.

9.7 Working Stress Design of Masonry

9.7.1 General

9.7.1.1 Scope. The design of masonry structures using working stress design shall comply with the provisions of Section 9.6 and this section. Stresses in clay or concrete masonry under service loads shall not exceed the values given in this section.

9.7.1.2 Allowable masonry stresses. When quality assurance provisions do not include requirements for special inspection as prescribed in Section 6.1, the allowable stresses for masonry in Section 9.7 shall be reduced by one half.

When one half allowable masonry stresses are used in Seismic Zones 3 and 4, the value of f'_m from Table 9.4 shall be limited to a maximum of 10 MPa (1,500 psi) for concrete masonry and 18 MPa (2,600 psi) for clay masonry unless the value of f'_m is verified by tests in accordance with Section 9.5.3.4, Items 1 and 4 or 6. A letter of certification is not required.

When one half allowable masonry stresses are used for design in Seismic Zones 3 and 4, the value of f'_m shall be limited to 10 MPa (1,500 psi) for concrete masonry and 18 MPa (2,600 psi) for clay masonry for Section 9.5.3.2, Item 3, and Section 9.5.3.3, Item 5, unless the value of f'_m is verified during construction by the testing requirements of Section 9.5.3.2, Item 2. A letter of certification is not required.

9.7.1.3 Minimum dimensions for masonry structures located in Seismic Zones 3 and 4. Elements of masonry structures located in Seismic Zones 3 and 4 shall be in accordance with this section.

9.7.1.3.1 Bearing walls. The nominal thickness of reinforced masonry bearing walls shall not be less than 150 mm (6 inches) except that nominal 102 mm (4 inch) thick load-bearing reinforced hollow-clay unit masonry walls may be used, provided net area unit strength exceeds 55 MPa (8,000 psi), units are laid in running bond, bar sizes do not exceed 12 mm (1/2 inch) with no more than two bars or one splice in a cell, and joints are flush cut, concave or a protruding V section.

9.7.1.3.2 Columns. The least nominal dimension of a reinforced masonry column shall be 305 mm (12 inches) except that, for working stress design, if the allowable stresses are reduced by one half, the minimum nominal dimension shall be 200 mm (8 inches).

9.7.1.4 Design assumptions. The working stress design procedure is based on working stresses and linear stress-strain distribution assumptions with all stresses in the elastic range as follows:

1. Plane sections before bending remain plane after bending.
2. Stress is proportional to strain.
3. Masonry elements combine to form a homogenous member.

9.7.1.5 Embedded anchor bolts

9.7.1.5.1 General. Allowable loads for plate anchor bolts, headed anchor bolts and bent bar anchor bolts shall be determined in accordance with this section.

9.7.1.5.2 Tension. Allowable loads in tension shall be the lesser value selected from Tables 9.5-1 and 9.5-2 or shall be determined from the lesser of Formula (9.7-1) or Formula (9.7-2).

$$B_t = 0.042 A_p \sqrt{f'_m} \quad (9.7-1)$$

For FPS:

$$B_t = 0.5 A_p \sqrt{f'_m}$$

$$B_t = 0.2 A_b f_y \quad (9.7-2)$$

The area A_p shall be the lesser of Formula (9.7-3) or Formula (9.7-4) and where the projected areas of adjacent anchor bolts overlap, A_p of each anchor bolt shall be reduced by one half of the overlapping area.

$$A_p = \pi l_b^2 \quad (9.7-3)$$

$$A_p = \pi l_{be}^2 \quad (9.7-4)$$

9.7.1.5.3 Shear. Allowable loads in shear shall be the value selected from Table 9.6 or shall be determined from the lesser of Formula (9.7-5) or Formula (9.7-6).

$$B_v = 1070 \sqrt[4]{f'_m A_b} \quad (9.7-5)$$

For FPS:

$$B_v = 350 \sqrt[4]{f'_m A_b}$$

$$B_v = 0.12 A_b f_y \quad (9.7-6)$$

Where the anchor bolt edge distance l_{be} in the direction of load is less than 12 bolt diameters, the value of B_v in Formula (9.7-5) shall be reduced by linear interpolation to zero at an l_{be} distance of 40 mm (1.5 inches). Where adjacent anchors are spaced closer than $8d_b$, the allowable shear of the adjacent anchors determined by Formula (9.7-5) shall be reduced by linear interpolation to 0.75 times the allowable shear value at a center-to-center spacing of four bolt diameters.

9.7.1.5.4 Combined shear and tension. Anchor bolts subjected to combined shear and tension shall be designed in accordance with Formula (9.7-7).

$$\frac{b_t}{B_t} + \frac{b_v}{B_v} \leq 1.0 \quad (9.7-7)$$

9.7.1.6 Compression in walls and columns

9.7.1.6.1 Walls, axial loads. Stresses due to compressive forces applied at the centroid of wall may be computed by Formula (9.7-8) assuming uniform distribution over the effective area.

$$f_a = \frac{P}{A_e} \quad (9.7-8)$$

9.7.1.6.2 Columns, axial loads. Stresses due to compressive forces applied at the centroid of columns may be computed by Formula (9.7-8) assuming uniform distribution over the effective area.

9.7.1.6.3 Columns, bending or combined bending and axial loads. Stresses in columns due to combined bending and axial loads shall satisfy the requirements of Section 9.7.2.7 where f_a/F_a is replaced by P/P_a . Columns subjected to bending shall meet all applicable requirements for flexural design.

9.7.1.7 Shear walls, design loads. When calculating shear or diagonal tension stresses, shear walls which resist seismic forces in Seismic Zones 3 and 4 shall be designed to resist 1.5 times the forces required by Section 5.30.

9.7.1.8 Design, Composite construction

9.7.1.8.1 General. The requirements of this section govern multiwythe masonry in which at least one wythe has strength or composition characteristics different from the other wythe or wythes and is adequately bonded to act as a single structural element.

The following assumptions shall apply to the design of composite masonry:

1. Analysis shall be based on elastic transformed section of the net area.
2. The maximum computed stress in any portion of composite masonry shall not exceed the allowable stress for the material of that portion.

9.7.1.8.2 Determination of moduli of elasticity. The modulus of elasticity of each type of masonry in composite construction shall be measured by tests if the modular ratio of the respective types of masonry exceeds 2 to 1 as determined by Section 9.6.2.12.

9.7.1.8.3 Structural continuity

9.7.1.8.3.1 Bonding of wythes. All wythes of composite masonry elements shall be tied together as specified in Section 9.6.1.5.2 as a minimum requirement. Additional ties or the combination of grout and metal ties shall be provided to transfer the calculated stress.

9.7.1.8.3.2 Material properties. The effect of dimensional changes of the various materials and different boundary conditions of various wythes shall be included in the design.

9.7.1.8.4 Design procedure, transformed sections. In the design of transformed sections, one material is chosen as the reference material, and the other materials are transformed to an equivalent area of the reference material by multiplying the areas of the other materials by the respective ratios of the moduli of elasticity of the other materials to that of the reference material. Thickness of the transformed area and its distance perpendicular to a given bending axis remain unchanged. Effective height or length of the element remains unchanged.

9.7.1.9 Reuse of masonry units. The allowable working stresses for reused masonry units shall not exceed 50 percent of those permitted for new masonry units of the same properties.

9.7.2 Design of Reinforced Masonry

9.7.2.1 Scope. The requirements of this section are in addition to the requirements of Sections 9.6 and 9.7.1, and govern masonry in which reinforcement is used to resist forces.

9.7.2.2 Reinforcement

9.7.2.2.1 Maximum reinforcement size. The maximum size of reinforcement shall be 32 mm (No. 11 bars). Maximum reinforcement area in cells shall be 6 percent of the cell area without splices and 12 percent of the cell area with splices.

9.7.2.2.2 Cover. All reinforcing bars, except joint reinforcement, shall be completely embedded in mortar or grout and have a minimum cover, including the masonry unit, of at least 20 mm (3/4 inch), 40 mm (1.5 inches) of cover when the masonry is exposed to weather and 50 mm (2 inches) of cover when the masonry is exposed to soil.

9.7.2.2.3 Development length. The required development length l_d for deformed bars or deformed wire shall be calculated by:

$$l_d = 0.29d_b f_s \quad \text{for bars in tension} \quad (9.7-9)$$

For FPS:

$$l_d = 0.002d_b f_s \quad \text{for bars in tension}$$

$$l_d = 0.22d_b f_s \quad \text{for bars in compression} \quad (9.7-10)$$

For FPS:

$$l_d = 0.0015d_b f_s \quad \text{for bars in compression}$$

Development length for smooth bars shall be twice the length determined by Formula (9.7-9).

9.7.2.2.4 Reinforcement bond stress. Bond stress u in reinforcing bars shall not exceed the following:

Plain Bars 413 kPa (60 psi)

Deformed Bars 1378 kPa (200 psi)

Deformed Bars without Special Inspection 689 kPa (100 psi)

9.7.2.2.5 Hooks

1. The term "standard hook" shall mean one of the following:
 - 1.1 A 180-degree turn plus extension of at least four bar diameters, but not less than 60 mm (2.5 inches) at free end of bar.
 - 1.2 A 90-degree turn plus extension of at least 12 bar diameters at free end of bar.
 - 1.3 For stirrup and tie anchorage only, either a 90-degree or a 135-degree turn, plus an extension of at least six bar diameters, but not less than 60 mm (2.5 inches) at the free end of the bar.
2. Inside diameter of bend of the bars, other than for stirrups and ties, shall not be less than that set forth in Table 9.7.
3. Inside diameter of bend for 166 mm (#5) or smaller stirrups and ties shall not be less than four bar diameters. Inside diameter of bend for 16mm (#5) or larger stirrups and ties shall not be less than that set forth in Table 9.7.
4. Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported end of continuous or restrained beams.
5. Hooks shall not be assumed to carry a load which would produce a tensile stress in the bar greater than 52 MPa (7,500 psi).
6. Hooks shall not be considered effective in adding to the compressive resistance of bars.
7. Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Data must be presented to show the adequacy of such devices.

9.7.2.2.6 Splices. The amount of lap of lapped splices shall be sufficient to transfer the allowable stress of the reinforcement as specified in Sections 9.6.3.4, 9.7.2.2.3 and 9.7.2.12. In no

case shall the length of the lapped splice be less than 30 bar diameters for compression or 40 bar diameters for tension. Welded or mechanical connections shall develop 125 percent of the specified yield strength of the bar in tension.

Exception: For compression bars in columns that are not part of the seismic-resisting system and are not subject to flexure, only the compressive strength need be developed.

When adjacent splices in grouted masonry are separated by 75 mm (3 inches) or less, the required lap length shall be increased 30 percent.

Exception: Where lap splices are staggered at least 24 bar diameters, no increase in lap length is required. See Section 9.7.2.12 for lap splice increases.

9.7.2.3 Design assumptions. The following assumptions are in addition to those stated in Section 9.7.1.4:

1. Masonry carries no tensile stress.
2. Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogenous material within the range of allowable working stresses.

9.7.2.4 Nonrectangular flexural elements. Flexural elements of nonrectangular cross section shall be designed in accordance with the assumptions given in Sections 9.7.1.4 and 9.7.2.3.

9.7.2.5 Allowable axial compressive stress and force. For members other than reinforced masonry columns, the allowable axial compressive stress F_a shall be determined as follows:

$$F_a = 0.25 f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad \text{for } h'/r \leq 99 \quad (9.7-11)$$

$$F_a = 0.25 f'_m \left(\frac{70r}{h'} \right)^2 \quad \text{for } h'/r > 99 \quad (9.7-12)$$

For reinforced masonry columns, the allowable axial compressive force P_a shall be determined as follows:

$$P_a = [0.25 f'_m A_e + 0.65 A_s F_{sc}] \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad \text{for } h'/r < 99 \quad (9.7-13)$$

$$P_a = [0.25 f'_m A_e + 0.65 A_s F_{sc}] \left(\frac{70r}{h'} \right)^2 \quad \text{for } h'/r > 99 \quad (9.7-14)$$

9.7.2.6 Allowable flexural compressive stress. The allowable flexural compressive stress F_b is:

$$F_b = 0.33 f'_m, \quad 13.8 \text{ MPa} \quad (2,000 \text{ psi}) \text{ maximum} \quad (9.7-15)$$

9.7.2.7 Combined compressive stresses, unity formula. Elements subjected to combined axial and flexural stresses shall be designed in accordance with accepted principles of mechanics or in accordance with Formula (9.7-16):

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad (9.7-16)$$

9.7.2.8 Allowable shear stress in flexural members. Where no shear reinforcement is provided, the allowable shear stress F_v in flexural members is:

$$F_v = 0.083 \sqrt{f'_m}, \quad 345 \text{ kPa maximum} \quad (9.7-17)$$

For FPS:

$$F_v = 1.0 \sqrt{f'_m}, \quad 50 \text{ psi maximum}$$

Exception: For a distance of 1/16 the clear span beyond the point of inflection, the maximum stress shall be 140 kPa (20 psi). Where shear reinforcement designed to take entire shear force is provided, the allowable shear stress F_v in flexural members is:

$$F_v = 0.25 \sqrt{f'_m}, \quad 1.0 \text{ MPa maximum} \quad (9.7-18)$$

For FPS:

$$F_v = 3.0 \sqrt{f'_m}, \quad 150 \text{ psi maximum}$$

9.7.2.9 Allowable shear stress in shear walls. Where in plane flexural reinforcement is provided and masonry is used to resist all shear, the allowable shear stress F_v in shear walls is:

$$\text{For } \frac{M}{Vd} < 1 \\ F_v = \frac{1}{36} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}, \quad \left(80 - 45 \frac{M}{Vd} \right) \text{ maximum} \quad (9.7-19)$$

For FPS:

$$F_v = \frac{1}{3} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}, \quad \left(80 - 45 \frac{M}{Vd} \right) \text{ maximum}$$

$$\text{For } \frac{M}{Vd} \geq 1, \quad F_v = \frac{1}{12} \sqrt{f'_m}, \quad 240 \text{ kPa maximum} \quad (9.7-20)$$

For FPS:

$$F_v = 1.0 \sqrt{f'_m}, \quad 35 \text{ psi maximum}$$

Where shear reinforcement designed to take all the shear is provided, the allowable shear stress F_v in shear walls is:

$$\text{For } \frac{M}{Vd} < 1 \\ F_v = \frac{1}{24} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}, \quad \left(120 - 45 \frac{M}{Vd} \right) \text{ maximum} \quad (9.7-21)$$

For FPS:

$$F_v = \frac{1}{2} \left(4 - \frac{M}{Vd} \right) \sqrt{f'_m}, \quad \left(120 - 45 \frac{M}{Vd} \right) \text{ maximum}$$

$$\text{For } \frac{M}{Vd} \geq 1$$

$$F_v = 0.12 \sqrt{f'_m}, \quad 520 \text{ kPa maximum} \quad (9.7-22)$$

For FPS:

$$F_v = 1.5\sqrt{f'_m} , \quad 75 \text{ psi maximum}$$

9.7.2.10 Allowable bearing stress. When a member bears on the full area of a masonry element, the allowable bearing stress F_{br} is:

$$F_{br} = 0.26 f'_m \quad (9.7-23)$$

When a member bears on one third or less of a masonry element, the allowable bearing stress F_{br} is:

$$F_{br} = 0.38 f'_m \quad (9.7-24)$$

Formula (9.7-24) applies only when the least dimension between the edges of the loaded and unloaded areas is a minimum of one fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one third but less than the full area shall be interpolated between the values of Formulas (9.7-23) and (9.7-24).

9.7.2.11 Allowable stresses in reinforcement. The allowable stresses in reinforcement shall be as follows:

1. *Tensile stress.*

1.1 Deformed bars,

$$F_s = 0.50 f_y , 165 \text{ MPa (24,000 psi) maximum} \quad (9.7-25)$$

1.2 Wire reinforcement,

$$F_s = 0.50 f_y , 207 \text{ MPa (30,000 psi) maximum} \quad (9.7-26)$$

1.3 Ties, anchors and smooth bars,

$$F_s = 0.40 f_y , 138 \text{ MPa (20,000 psi) maximum} \quad (9.7-27)$$

2. *Compressive stress.*

2.1 Deformed bars in columns,

$$F_{sc} = 0.40 f_y , 165 \text{ MPa (24,000 psi) maximum} \quad (9.7-28)$$

2.2 Deformed bars in flexural members,

$$F_{sc} = 0.50 f_y , 165 \text{ MPa (24,000 psi) maximum} \quad (9.7-29)$$

2.3 Deformed bars in shear walls which are confined by lateral ties throughout the distance where compression reinforcement is required and where such lateral ties are not less than 1/4 inch in diameter and spaced not farther apart than 16 bar diameters or 48 tie diameters,

$$F_{sc} = 0.40 f_y , 165 \text{ MPa (24,000 psi) maximum} \quad (9.7-30)$$

9.7.2.12 Lap splice increases. In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tensile stress F_s , the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase may be used.

9.7.2.13 Reinforcement for columns. Columns shall be provided with reinforcement as specified in this section.

9.7.2.13.1 Vertical reinforcement. The area of vertical reinforcement shall not be less than $0.005 A_e$ and not more than $0.04 A_e$. At least four No. 3 bars shall be provided. The minimum clear distance between parallel bars in columns shall be two and one half times the bar diameter.

9.7.2.14 Compression in walls and columns

9.7.2.14.1 General. Stresses due to compressive forces in walls and columns shall be calculated in accordance with Section 9.7.2.5.

9.7.2.14.2 Walls, bending or combined bending and axial loads. Stresses in walls due to combined bending and axial loads shall satisfy the requirements of Section 9.7.2.7 where f_a is given by Formula (9.7-8). Walls subjected to bending with or without axial loads shall meet all applicable requirements for flexural design.

The design of walls with an h'/t ratio larger than 30 shall be based on forces and moments determined from an analysis of the structure. Such analysis shall consider the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, effect of deflections on moments and forces and the effects of duration of loads.

9.7.2.15 Flexural design, rectangular flexural elements. Rectangular flexural elements shall be designed in accordance with the following formulas or other methods based on the assumptions given in Sections 9.7.1.4, 9.7.2.3 and this section.

1. *Compressive stress in the masonry:*

$$F_b = \frac{M}{bd^2} \left(\frac{2}{jk} \right) \quad (9.7-31)$$

2. *Tensile stress in the longitudinal reinforcement:*

$$F_s = \frac{M}{A_s jd} \quad (9.7-32)$$

3. *Design coefficients:*

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho \quad (9.7-33)$$

$$k = \frac{1}{1 + \frac{f_s}{nf_b}} \quad (9.7-34)$$

$$j = 1 - \frac{k}{3} \quad (9.7-35)$$

9.7.2.16 Bond of flexural reinforcement. In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress shall be computed by the formula:

$$u = \frac{V}{\sum_o jd} \quad (9.7-36)$$

9.7.2.17 Shear in flexural members and shear walls. The shear stress in flexural members and shear walls shall be computed by:

$$f_v = \frac{V}{bjd} \quad (9.7-37)$$

For members of *T* or *I* section, b' shall be substituted for b . Where f_v as computed by Formula (9.7-37) exceeds the allowable shear stress in masonry, F_v , web reinforcement shall be provided and designed to carry the total shear force. Both vertical and horizontal shear stresses shall be considered. The area required for shear reinforcement placed perpendicular to the longitudinal reinforcement shall be computed by:

$$A_v = \frac{sV}{F_s d} \quad (9.7-38)$$

Where web reinforcement is required, it shall be so spaced that every 45-degree line extending from a point at $d/2$ of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement.

9.7.3 Design of Unreinforced Masonry

9.7.3.1 General. The requirements of this section govern masonry in which reinforcement is not used to resist design forces and are in addition to the requirements of Sections 9.6 and 9.7.1.

9.7.3.2 Allowable axial compressive stress. The allowable axial compressive stress F_a is:

$$F_a = 0.25 f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] \quad \text{for } h'/r \leq 99 \quad (9.7-39)$$

$$F_a = 0.25 f'_m \left(\frac{70r}{h'} \right)^2 \quad \text{for } h'/r > 99 \quad (9.7-40)$$

9.7.3.3 Allowable flexural compressive stress. The allowable flexural compressive stress F_b is:

$$F_b = 0.33 f'_m, \quad 13.8 \text{ MPa (2000 psi) maximum} \quad (9.7-41)$$

9.7.3.4 Combined compressive stresses, unity formula. Elements subjected to combined axial and flexural stresses shall be designed in accordance with accepted principles of mechanics or in accordance with the Formula (9.7-42):

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \quad (9.7-42)$$

9.7.3.5 Allowable tensile stress. Resultant tensile stress due to combined bending and axial load shall not exceed the allowable flexural tensile stress, F_t .

The allowable tensile stress for walls in flexure without tensile reinforcement using portland cement and hydrated lime, or using mortar cement Type M or S mortar, shall not exceed the values in Table 9.11.

Values in Table 9.11 for tension normal to head joints are for running bond; no tension is allowed across head joints in stack bond masonry. These values shall not be used for horizontal flexural members.

9.7.3.6 Allowable shear stress in flexural members. The allowable shear stress F_v in flexural members is:

$$F_v = 0.083 \sqrt{f'_m} , \text{ 345 kPa maximum} \quad (9.7-43)$$

For FPS:

$$F_v = 1.0 \sqrt{f'_m} , \text{ 50 psi maximum}$$

Exception: For a distance of 1/16th the clear span beyond the point of inflection, the maximum stress shall be 138 kPa (20 psi).

9.7.3.7 Allowable shear stress in shear walls. The allowable shear stress F_v in shear walls is as follows:

1. Clay units $F_v = 0.025 f'_m$, 551 kPa maximum

For FPS:

$$F_v = 0.3 f'_m , \text{ 80 psi maximum}$$

2. Concrete units with Type M or S mortar, $F_v = 234$ kPa (34 psi) maximum.

3. Concrete units with Type N mortar, $F_v = 158$ kPa (23 psi) maximum.

4. The allowable shear stress in un-reinforced masonry may be increased by $0.2 f_{md}$.

9.7.3.8 Allowable bearing stress. When a member bears on the full area of a masonry element, the allowable bearing stress F_{br} shall be:

$$F_{br} = 0.26 f'_m \quad (9.7-45)$$

When a member bears on one-third or less of a masonry element, the allowable bearing stress F_{br} shall be:

$$F_{br} = 0.38 f'_m \quad (9.7-46)$$

Formula (9.7-46) applies only when the least dimension between the edges of the loaded and unloaded areas is a minimum of one fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one third but less than the full area shall be interpolated between the values of Formulas (9.7-45) and (9.7-46).

9.7.3.9 Combined bending and axial loads, compressive stresses. Compressive stresses due to combined bending and axial loads shall satisfy the requirements of Section 9.7.3.4.

9.7.3.10 Compression in walls and columns. Stresses due to compressive forces in walls and columns shall be calculated in accordance with Section 9.7.2.5.

9.7.3.11 Flexural design. Stresses due to flexure shall not exceed the values given in Sections 9.7.1.2, 9.7.3.3 and 9.7.3.5, where:

$$f_b = \frac{Mc}{I} \quad (9.7-47)$$

9.7.3.12 Shear in flexural members and shear walls. Shear calculations for flexural members and shear walls shall be based on Formula (9.7-48).

$$f_v = \frac{V}{A_e} \quad (9.7-48)$$

9.7.3.13 Corbels. The slope of corbelling (angle measured from the horizontal to the face of the corbelled surface) of unreinforced masonry shall not be less than 60 degrees. The maximum horizontal projection of corbelling from the plane of the wall shall be such that allowable stresses are not exceeded.

9.7.3.14 Stack bond. Masonry units laid in stack bond shall have longitudinal reinforcement of at least 0.00027 times the vertical cross-sectional area of the wall placed horizontally in the bed joints or in bond beams spaced vertically not more than 1200 mm (48 inches) apart.

9.8 Strength Design of Masonry

9.8.1 General

9.8.1.1 General provisions. The design of hollow-unit clay and concrete masonry structures using strength design shall comply with the provisions of Section 9.6 and this section.

Exception: Two-wythe solid-unit masonry may be used under Sections 9.8.2.1 and 9.8.2.4.

9.8.1.2 Special inspection during construction shall be provided as set forth in Section 6.1.5, Item 7.

9.8.1.3 Required strength. The required strength shall be determined in accordance with the factored load combinations of Section 5.12.2.

9.8.1.4 Design strength. Design strength is the nominal strength, multiplied by the strength-reduction factor, ϕ , as specified in this section. Masonry members shall be proportioned such that the design strength exceeds the required strength.

9.8.1.4.1 Beams, piers and columns

9.8.1.4.1.1 Flexure. Flexure with or without axial load, the value of ϕ shall be determined from Formula (9.8-1):

$$\phi = 0.8 - \frac{P_u}{A_e f'_m} \quad (9.8-1)$$

and $0.60 \leq \phi \leq 0.80$

9.8.1.4.1.2 Shear. Shear: $\phi = 0.60$

9.8.1.4.2 Wall design for out-of-plane loads

9.8.1.4.2.1 *Walls with unfactored axial load of $0.04 f'_m$ or less.* Flexure: $\phi = 0.80$.

9.8.1.4.2.2 *Walls with unfactored axial load greater than $0.04 f'_m$.* Axial load and axial load with flexure: $\phi = 0.80$. Shear: $\phi = 0.60$.

9.8.1.4.3 Wall design for in-plane loads

9.8.1.4.3.1 Axial load. Axial load and axial load with flexure: $\phi = 0.65$. For walls with symmetrical reinforcement in which f_y does not exceed 413 MPa (60,000 psi), the value of ϕ may be increased linearly to 0.85 as the value of ϕP_n decreases from $0.10 f'_m A_e$ or $0.25 P_b$ to zero. For solid grouted walls, the value of P_b may be calculated by Formula (9.8-2)

$$P_b = 0.85 f'_m b a_b \quad (9.8-2)$$

Where:

$$a_b = 0.85 d [e_{mu} / (e_{mu} + (f_y/E_s))] \quad (9.8-3)$$

9.8.1.4.3.2 Shear. Shear: $\phi = 0.60$. The value of ϕ may be 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored-load combination.

9.8.1.4.4 Moment-resisting wall frames

9.8.1.4.4.1 Flexure with or without axial load. The value of ϕ shall be as determined from Formula (9.8-4); however, the value of ϕ shall not be less than 0.65 nor greater than 0.85.

$$\phi = 0.85 - 2 \left(\frac{P_u}{A_e f'_m} \right) \quad (9.8-4)$$

9.8.1.4.4.2 Shear. Shear: $\phi = 0.80$.

9.8.1.4.5 Anchor. Anchor: $\phi = 0.80$.

9.8.1.4.6 Reinforcement

9.8.1.4.6.1 Development. Development: $\phi = 0.80$.

9.8.1.4.6.2 Splices. Splices: $\phi = 0.80$.

9.8.1.5 Anchor bolts

9.8.1.5.1 Required strength. The required strength of embedded anchor bolts shall be determined from factored loads as specified in Section 9.8.1.3.

9.8.1.5.2 Nominal anchor bolt strength. The nominal strength of anchor bolts times the strength-reduction factor shall equal or exceed the required strength. The nominal tensile capacity of anchor bolts shall be determined from the lesser of Formula (9.8-5) or (9.8-6).

$$B_m = 0.084 A_p \sqrt{f'_m} \quad (9.8-5)$$

For FPS:

$$B_m = 1.0 A_p \sqrt{f'_m}$$

$$B_m = 0.4 A_b f_y \quad (9.8-6)$$

The area A_p shall be the lesser of Formula (9.8-7) or (9.8-8) and where the projected areas of adjacent anchor bolts overlap, the value of A_p of each anchor bolt shall be reduced by one half of the overlapping area.

$$A_p = \pi l_b^2 \quad (9.8-7)$$

$$A_p = \pi l_{be}^2 \quad (9.8-8)$$

The nominal shear capacity of anchor bolts shall be determined from the lesser of Formula (9.8-9) or (9.8-10).

$$B_{sn} = 2750 \sqrt[4]{f'_m A_b} \quad (9.8-9)$$

For FPS:

$$B_{sn} = 900 \sqrt[4]{f'_m A_b}$$

$$B_{sn} = 0.25 A_b f_y \quad (9.8-10)$$

Where the anchor bolt edge distance, l_{be} , in the direction of load is less than 12 bolt diameters, the value of B_{sn} in Formula (9.8-9) shall be reduced by linear interpolation to zero at an l_{be} distance of 40 mm (1.5 inches). Where adjacent anchor bolts are spaced closer than $8d_b$, the nominal shear strength of the adjacent anchors determined by Formula (9.8-9) shall be reduced by linear interpolation to 0.75 times the nominal shear strength at a center-to-center spacing of four bolt diameters. Anchor bolts subjected to combined shear and tension shall be designed in accordance with Formula (9.8-11).

$$\frac{b_{tu}}{\phi B_{tn}} + \frac{b_{su}}{\phi B_{sn}} \leq 1.0 \quad (9.8-11)$$

9.8.1.5.3 Anchor bolt placement. Anchor bolts shall be placed so as to meet the edge distance, embedment depth and spacing requirements of Sections 9.6.2.14.2, 9.6.2.14.3 and 9.6.2.14.4.

9.8.2 Reinforced Masonry

9.8.2.1 General

9.8.2.1.1 Scope. The requirements of this section are in addition to the requirements of Sections 9.6 and 9.8.1 and govern masonry in which reinforcement is used to resist forces.

9.8.2.1.2 Design assumptions. The following assumptions apply:

Masonry carries no tensile stress greater than the modulus of rupture. Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material.

Nominal strength of singly reinforced masonry wall cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains.

Strain in reinforcement and masonry walls shall be assumed to be directly proportional to the distance from the neutral axis.

Maximum usable strain, e_{mu} , at the extreme masonry compression fiber shall:

1. Be 0.003 for the design of beams, piers, columns and walls.
2. Not exceed 0.003 for moment-resisting wall frames, unless lateral reinforcement as defined in Section 9.8.2.6.2.6 is utilized.

Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis. Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain.

For strains greater than that corresponding to f_y , stress in reinforcement shall be considered independent of strain and equal to f_y . Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except when computing requirements for deflection. Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

Masonry stress of $0.85 f'_m$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fiber of maximum compressive strain. Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

9.8.2.2 Reinforcement requirements and details

9.8.2.2.1 Maximum reinforcement. The maximum size of reinforcement shall be 288 mm diameter (#9). The diameter of a bar shall not exceed one fourth the least dimension of a cell. No more than two bars shall be placed in a cell of a wall or a wall frame.

9.8.2.2.2 Placement. The placement of reinforcement shall comply with the following: In columns and piers, the clear distance between vertical reinforcing bars shall not be less than one and one-half times the nominal bar diameter, nor less than 40 mm (1.5 inches).

9.8.2.2.3 Cover. All reinforcing bars shall be completely embedded in mortar or grout and shall have a cover of not less than 40 mm (1.5 inches) nor less than $2.5 d_b$.

9.8.2.2.4 Standard hooks. A standard hook shall be one of the following:

1. A 180-degree turn plus an extension of at least four bar diameters, but not less than 63 mm (2.5 inches) at the free end of the bar.
2. A 135-degree turn plus an extension of at least six bar diameters at the free end of the bar.
3. A 90-degree turn plus an extension of at least 12 bar diameters at the free end of the bar.

9.8.2.2.5 Minimum bend diameter for reinforcing bars. Diameter of bend measured on the inside of a bar other than for stirrups and ties in sizes 10 mm diameter (#3) through 16 mm diameter (#5) shall not be less than the values in Table 9.7. Inside diameter of bends for stirrups and ties shall not be less than $4d_b$ for 16 mm (#5) bars and smaller. For bars larger than 16mm (#5), diameter of bend shall be in accordance with Table 9.7.

9.8.2.2.6 Development. The calculated tension or compression reinforcement shall be developed in accordance with the following provisions:

The embedment length of reinforcement shall be determined by Formula (9.8-12).

$$l_d = l_{de}/\phi \quad (9.8-12)$$

Where:

$$l_d = \frac{1.8 d_b^2 f_y}{K \sqrt{f'_m}} \leq 52 d_b \quad (9.8-13)$$

For FPS:

$$l_d = \frac{0.15 d_b^2 f_y}{K \sqrt{f'_m}} \leq 52 d_b$$

K shall not exceed $3d_b$. The minimum embedment length of reinforcement shall be 300 mm (12 inches).

9.8.2.2.7 *Splices.* Reinforcement splices shall comply with one of the following:

1. The minimum length of lap for bars shall be 300 mm (12 inches) or the length determined by Formula (9.8-14).

$$l_d = l_{de}/\phi \quad (9.8-14)$$

Bars spliced by non contact lap splices shall be spaced transversely a distance not greater than one fifth the required length of lap or more than 200 mm (8 inches).

2. A welded splice shall have the bars butted and welded to develop in tension 125 percent of the yield strength of the bar, f_y .
3. Mechanical splices shall have the bars connected to develop in tension or compression, as required, at least 125 percent of the yield strength of the bar, f_y .

9.8.2.3 *Design of beams, piers and columns*

9.8.2.3.1 General. The requirements of this section are for the design of masonry beams, piers and columns. The value of f'_m shall not be less than 10 MPa (1,500 psi). For computational purposes, the value of f'_m shall not exceed 28 MPa (4,000 psi).

9.8.2.3.2 Design assumptions. Member design forces shall be based on an analysis which considers the relative stiffness of structural members. The calculation of lateral stiffness shall include the contribution of all beams, piers and columns. The effects of cracking on member stiffness shall be considered. The drift ratio of piers and columns shall satisfy the limits specified in Chapter 16 of UBC 97.

9.8.2.3.3 Balanced reinforcement ratio for compression limit state. Calculation of the balanced reinforcement ratio, ρ_b , shall be based on the following assumptions:

1. The distribution of strain across the section shall be assumed to vary linearly from the maximum usable strain, e_{mu} , at the extreme compression fiber of the element, to a yield strain of f_y/E_s at the extreme tension fiber of the element.
2. Compression forces shall be in equilibrium with the sum of tension forces in the reinforcement and the maximum axial load associated with a loading combination $1.0D + 1.0L + (1.4E \text{ or } 1.3W)$.
3. The reinforcement shall be assumed to be uniformly distributed over the depth of the element and the balanced reinforcement ratio shall be calculated as the area of this reinforcement divided by the net area of the element.

4. All longitudinal reinforcement shall be included in calculating the balanced reinforcement ratio except that the contribution of compression reinforcement to resistance of compressive loads shall not be considered.

9.8.2.3.4 Required strength. Except as required by Sections 9.8.2.3.6 through 9.8.2.3.12, the required strength shall be determined in accordance with Section 9.8.1.3.

9.8.2.3.5 Design strength. Design strength provided by beam, pier or column cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 9.8.1.4.

9.8.2.3.6 Nominal strength

9.8.2.3.6.1 Nominal axial and flexural strength. The nominal axial strength, P_n , and the nominal flexural strength, M_n , of a cross section shall be determined in accordance with the design assumptions of Section 9.8.2.1.2 and 9.8.2.3.2. The maximum nominal axial compressive strength shall be determined in accordance with Formula (9.8-15).

$$P_n = 0.80 \left[0.85 f'_m (A_e - A_s) + f_y A_s \right] \quad (9.8-15)$$

9.8.2.3.6.2 Nominal shear strength. The nominal shear strength shall be determined in accordance with Formula (9.8-16).

$$V_n = V_m + V_s \quad (9.8-16)$$

Where:

$$V_m = 0.083 C_d A_e \sqrt{f'_m} , \quad 63 C_d A_e \text{ maximum} \quad (9.8-17)$$

For FPS:

$$V_m = C_d A_e \sqrt{f'_m} , \quad 63 C_d A_e \text{ maximum}$$

and

$$V_s = A_e \rho_n f_y \quad (9.8-18)$$

1. The nominal shear strength shall not exceed the value given in Table 9.12.
2. The value of V_m shall be assumed to be zero within any region subjected to net tension factored loads.
3. The value of V_m shall be assumed to be 172 kPa (25 psi) where M_u is greater than 0.7 M_n . The required moment, M_u , for seismic design for comparison with the 0.7 M_n value of this section shall be based on an R of 3.

9.8.2.3.7 Reinforcement

1. Where transverse reinforcement is required, the maximum spacing shall not exceed one half the depth of the member nor 1200 mm (48 inches).
2. Flexural reinforcement shall be uniformly distributed throughout the depth of the element.
3. Flexural elements subjected to load reversals shall be symmetrically reinforced.
4. The nominal moment strength at any section along a member shall not be less than one fourth of the maximum moment strength.

5. The flexural reinforcement ratio, ρ , shall not exceed $0.5 \rho_b$.
6. Lap splices shall comply with the provisions of Section 9.8.2.2.7.
7. Welded splices and mechanical splices which develop at least 125 percent of the specified yield strength of a bar may be used for splicing the reinforcement. Not more than two longitudinal bars shall be spliced at a section. The distance between splices of adjacent bars shall be at least 750 mm (30 inches) along the longitudinal axis.
8. Specified yield strength of reinforcement shall not exceed 413 MPa (60,000 psi). The actual yield strength based on mill tests shall not exceed 1.3 times the specified yield strength.

9.8.2.3.8 Seismic design provisions. The lateral seismic load resistance in any line or storey level shall be provided by shear walls or wall frames, or a combination of shear walls and wall frames. Shear walls and wall frames shall provide at least 80 percent of the lateral stiffness in any line or storey level.

Exception: Where seismic loads are determined based on R not greater than 2 and where all joints satisfy the provisions of Section 9.8.2.6.2.9, the piers may be used to provide seismic load resistance.

9.8.2.3.9 Dimensional Limits. Dimensions shall be in accordance with the following:

1. *Beams.*

- 1.1 The nominal width of a beam shall not be less than 150 mm (6 inches).
- 1.2 The clear distance between locations of lateral bracing of the compression side of the beam shall not exceed 32 times the least width of the compression area.
- 1.3 The nominal depth of a beam shall not be less than 200 mm (8 inches).

2. *Piers.*

- 2.1 The nominal width of a pier shall not be less than 150 mm (6 inches) and shall not exceed 400 mm (16 inches).
- 2.2 The distance between lateral supports of a pier shall not exceed 30 times the nominal width of the piers except as provided for in Section 9.8.2.3.9, Item 2.3.
- 2.3 When the distance between lateral supports of a pier exceeds 30 times the nominal width of the pier, the provisions of Section 9.8.2.4 shall be used for design.
- 2.4 The nominal length of a pier shall not be less than three times the nominal width of the pier. The nominal length of a pier shall not be greater than six times the nominal width of the pier. The clear height of a pier shall not exceed five times the nominal length of the pier.

Exception: The length of a pier may be equal to the width of the pier when the axial force at the location of maximum moment is less than $0.04 f'_m A_g$.

3. *Columns.*

- 3.1 The nominal width of a column shall not be less than 300 mm (12 inches).
- 3.2 The distance between lateral supports of a column shall not exceed 30 times the nominal width of the column.
- 3.3 The nominal length of a column shall not be less than 300 mm (12 inches) and not greater than three times the nominal width of the column.

9.8.2.3.10 Beams

9.8.2.3.10.1 Scope. Members designed primarily to resist flexure shall comply with the requirements of this section. The factored axial compressive force on a beam shall not exceed $0.05 A_e f'_{m}$.

9.8.2.3.10.2 Longitudinal reinforcement

1. The variation in the longitudinal reinforcing bars shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.
2. The nominal flexural strength of a beam shall not be less than 1.3 times the nominal cracking moment strength of the beam. The modulus of rupture, f_r , for this calculation shall be assumed to be 1.6 MPa (235 psi).

9.8.2.3.10.3 Transverse reinforcement. Transverse reinforcement shall be provided where V_u exceeds V_m . Required shear, V_u , shall include the effects of drift. The value of V_u shall be based on ΔM . When transverse shear reinforcement is required, the following provisions shall apply:

1. Shear reinforcement shall be a single bar with a 180-degree hook at each end.
2. Shear reinforcement shall be hooked around the longitudinal reinforcement.
3. The minimum transverse shear reinforcement ratio shall be 0.0007.
4. The first transverse bar shall not be more than one fourth of the beam depth from the end of the beam.

9.8.2.3.10.4 Construction. Beams shall be solid grouted.

9.8.2.3.11 Piers

9.8.2.3.11.1 Scope. Piers proportioned to resist flexure and shear in conjunction with axial load shall comply with the requirements of this section. The factored axial compression on the piers shall not exceed $0.3 A_e f'_{m}$.

9.8.2.3.11.2 Longitudinal reinforcement. A pier subjected to in-plane stress reversals shall be longitudinally reinforced symmetrically on both sides of the neutral axis of the pier.

1. One bar shall be provided in the end cells.
2. The minimum longitudinal reinforcement ratio shall be 0.0007.

9.8.2.3.11.3 Transverse reinforcement. Transverse reinforcement shall be provided where V_u exceeds V_m . Required shear, V_u , shall include the effects of drift. The value of V_u shall be based on ΔM . When transverse shear reinforcement is required, the following provisions shall apply:

1. Shear reinforcement shall be hooked around the extreme longitudinal bars with a 180-degree hook. Alternatively, at wall intersections, transverse reinforcement with a 90-degree standard hook around a vertical bar in the intersecting wall shall be permitted.
2. The minimum transverse reinforcement ratio shall be 0.0015.

9.8.2.3.12 Columns

9.8.2.3.12.1 Scope. Columns shall comply with the requirements of this section.

9.8.2.3.12.2 Longitudinal reinforcement. Longitudinal reinforcement shall be a minimum of four bars, one in each corner of the column.

1. Maximum reinforcement area shall be $0.03 A_e$.

2. Minimum reinforcement area shall be $0.005 A_e$.

9.8.2.3.12.3 Lateral ties

1. Lateral ties shall be provided in accordance with Section 9.6.3.6.
2. Minimum lateral reinforcement area shall be $0.0018 A_g$.

9.8.2.3.12.4 Construction. Columns shall be solid grouted.

9.8.2.4 Wall design for out-of-plane loads

9.8.2.4.1 General. The requirements of this section are for the design of walls for out-of-plane loads.

9.8.2.4.2 Maximum reinforcement. The reinforcement ratio shall not exceed $0.5\rho_b$.

9.8.2.4.3 Moment and deflection calculations. All moment and deflection calculations in Section 9.8.2.4 are based on simple support conditions top and bottom. Other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

9.8.2.4.4 Walls with axial load of $0.04 f'_m$ or less. The procedures set forth in this section, which consider the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, shall be used when the vertical load stress at the location of maximum moment does not exceed $0.04 f'_m$ as computed by Formula (9.8-19). The value of f'_m shall not exceed 41 MPa (6,000 psi).

$$\frac{P_w + P_f}{A_g} \leq 0.04 f'_m \quad (9.8-19)$$

Walls shall have a minimum nominal thickness of 150 mm (6 inches). Required moment and axial force shall be determined at the mid height of the wall and shall be used for design. The factored moment, M_u , at the mid height of the wall shall be determined by Formula (9.8-20).

$$M_u = \frac{w_u h^2}{8} + P_{uf} \frac{e}{2} + P_u \Delta_u \quad (9.8-20)$$

Where:

Δ_u = deflection at midheight of wall due to factored loads

$$P_u = P_{uw} + P_{uf} \quad (9.8-21)$$

The design strength for out-of-plane wall loading shall be determined by Formula (9.8-22).

$$M_u \leq \phi M_n \quad (9.8-22)$$

Where:

$$M_n = A_{se} f_y (d - a/2) \quad (9.8-23)$$

$$A_{se} = (A_s f_y + P_u) / f_y, \text{ effective area of steel} \quad (9.8-24)$$

$$a = (P_u + A_s f_y) / 0.85 f'_m b, \text{ depth of stress block due to factored loads} \quad (9.8-25)$$

9.8.2.4.5 Wall with axial load greater than $0.04 f'_m$ The procedures set forth in this section shall be used for the design of masonry walls when the vertical load stresses at the location of maximum moment exceed $0.04 f'_m$ but are less than $0.2 f'_m$ and the slenderness ratio h/t does not exceed 30. Design strength provided by the wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 9.8.1.4. Walls shall be proportioned such that the design strength exceeds the required strength. The nominal shear strength shall be determined by Formula (9.8-26).

$$V_n = 0.166 A_{mv} \sqrt{f'_m} \quad (9.8-26)$$

For FPS:

$$V_n = 2 A_{mv} \sqrt{f'_m}$$

9.8.2.4.6 Deflection design. The mid height deflection, Δ_s , under service lateral and vertical loads (without load factors) shall be limited by the relation:

$$\Delta_s = 0.007 h \quad (9.8-27)$$

$P\Delta$ effects shall be included in deflection calculation. The mid height deflection shall be computed with the following formula:

$$\Delta_s = \frac{5M_s h^2}{48E_m I_g} \quad \text{for } M_{ser} \leq M_{cr} \quad (9.8-28)$$

$$\Delta_s = \frac{5M_s h^2}{48E_m I_g} + \frac{5(M_{ser} - M_{cr})h^2}{48E_m I_{cr}} \quad \text{for } M_{cr} < M_{ser} < M_n \quad (9.8-29)$$

The cracking moment strength of the wall shall be determined from the formula:

$$M_{cr} = Sf_r \quad (9.8-30)$$

The modulus of rupture, f_r , shall be as follows:

1. For fully grouted hollow-unit masonry,

$$f_r = 0.33 \sqrt{f'_m}, \quad 1.6 \text{ MPa maximum} \quad (9.8-31)$$

For FPS:

$$f_r = 4.0 \sqrt{f'_m}, \quad 235 \text{ psi maximum}$$

2. For partially grouted hollow-unit masonry,

$$f_r = 0.21 \sqrt{f'_m}, \quad 861 \text{ MPa maximum} \quad (9.8-32)$$

For FPS:

$$f_r = 2.5 \sqrt{f'_m}, \quad 125 \text{ psi maximum}$$

3. For two-wythe brick masonry,

$$f_r = 0.166 \sqrt{f'_m} \text{ 861 MPa maximum} \quad (9.8-33)$$

For FPS:

$$f_r = 2.0 \sqrt{f'_m} \text{ 125 psi maximum}$$

9.8.2.5 Wall design for in-plane loads

9.8.2.5.1 General. The requirements of this section are for the design of walls for in-plane loads. The value of f'_m shall not be less than 10 MPa (1,500 psi) nor greater than 28 MPa (4,000 psi).

9.8.2.5.2 Reinforcement. Reinforcement shall be in accordance with the following:

1. Minimum reinforcement shall be provided in accordance with Section 9.6.1.12.4, Item 2.3, for all seismic areas using this method of analysis.
2. When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.8 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall from Formula (9.8-30).
3. The amount of vertical reinforcement shall not be less than one half the horizontal reinforcement.
4. Spacing of horizontal reinforcement within the region defined in Section 9.8.2.5.5, Item 3, shall not exceed three times the nominal wall thickness nor 600 mm (24 inches).

9.8.2.5.3 Design strength. Design strength provided by the shear wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 9.8.1.4.3.

9.8.2.5.4 Axial strength. The nominal axial strength of the shear wall supporting axial loads only shall be calculated by Formula (9.8-34).

$$P_n = 0.85 f'_m (A_e - A_s) + f_y A_s \quad (9.8-34)$$

Axial design strength provided by the shear wall cross section shall satisfy Formula (9.8-35).

$$P_u \leq 0.80 \phi P_o \quad (9.8-35)$$

9.8.2.5.5 Shear strength. Shear strength shall be as follows:

1. The nominal shear strength shall be determined using either Item 2 or 3 below. Maximum nominal shear strength values are determined from Table 9.12.
2. The nominal shear strength of the shear wall shall be determined from Formula (9.8-36), except as provided in Item 3 below:

$$V_n = V_m + V_s \quad (9.8-36)$$

Where:

$$V_m = 0.083 C_d A_e \sqrt{f'_m} \quad (9.8-37)$$

For FPS:

$$V_m = C_d A_e \sqrt{f'_m}$$

and

$$V_s = A_{mv} \rho_n f_y \quad (9.8-38)$$

3. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist. For all cross sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined from Formula (9.8-39).

$$V_n = A_{mv} \rho_n f_y \quad (9.8-39)$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one half storey height. For the other region, the nominal shear strength of the shear wall shall be determined from Formula (9.8-36).

9.8.2.5.6 Boundary members. Boundary members shall be as follows:

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.0015. The strain shall be determined using factored forces and R equal to 1.5.
2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Section 9.8.2.6.2.7 is greater than 0.0015.
3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of 10 mm (#3) bars at a maximum of 200 mm (8-inch) spacing within the grouted core or equivalent confinement which can develop an ultimate compressive masonry strain of at least 0.006.

9.8.2.6 Design of moment-resisting wall frame

9.8.2.6.1 General requirements

9.8.2.6.1.1 Scope. The requirements of this section are for the design of fully grouted moment-resisting wall frames constructed of reinforced open-end hollow-unit concrete or hollow-unit clay masonry.

9.8.2.6.1.2 Dimensional limits. Dimensions shall be in accordance with the following.

Beams. Clear span for the beam shall not be less than two times its depth. The nominal depth of the beam shall not be less than two units or 400 mm (16 inches), whichever is greater. The nominal beam depth to nominal beam width ratio shall not exceed 6. The nominal width of the beam shall be the greater of 200 mm (8-inch) or 1/26 of the clear span between pier faces.

Piers. The nominal depth of piers shall not exceed 2400 mm (96 inches). Nominal depth shall not be less than two full units or 800 mm (32 inches), whichever is greater. The nominal width of piers shall not be less than the nominal width of the beam, nor less than 200 mm (8-inch) or 1/14 of the clear height between beam faces, whichever is greater. The clear height-to-depth ratio of piers shall not exceed 5.

9.8.2.6.1.3 Analysis. Member design forces shall be based on an analysis which considers the relative stiffness of pier and beam members, including the stiffening influence of joints. The calculation of beam moment capacity for the determination of pier design shall include any

contribution of floor slab reinforcement. The out-of-plane drift ratio of all piers shall satisfy the drift ratio limits specified in Section 5.30.10.2.

9.8.2.6.2 *Design procedure*

9.8.2.6.2.1 Required strength. Except as required by Sections 9.8.2.6.2.7 and 9.8.2.6.2.8, the required strength shall be determined in accordance with Section 9.8.1.3.

9.8.2.6.2.2 Design strength. Design strength provided by frame member cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ , specified in Section 9.8.1.4.4. Members shall be proportioned such that the design strength exceeds the required strength.

9.8.2.6.2.3 Design assumptions for nominal strength. The nominal strength of member cross sections shall be based on assumptions prescribed in Section 9.8.2.1.2.

The value of f'_{m} shall not be less than 10 MPa (1,500 psi) or greater than 28 MPa (4,000 psi).

9.8.2.6.2.4 Reinforcement. The nominal moment strength at any section along a member shall not be less than one fourth of the higher moment strength provided at the two ends of the member. Lap splices shall be as defined in Section 9.8.2.2.7. The center of the lap splice shall be at the center of the member clear length. Welded splices and mechanical connections may be used for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section, and the distance between splices of alternate bars is at least 610 mm (24 inches) along the longitudinal axis.

Reinforcement shall not have specified yield strength greater than 413 MPa (60,000 psi). The actual yield strength based on mill tests shall not exceed the specified yield strength times 1.3.

9.8.2.6.2.5 Flexural members (beams). Requirements of this section apply to beams proportioned primarily to resist flexure as follows:

The axial compressive force on beams due to factored loads shall not exceed $0.10 A_n f'_{m}$.

1. **Longitudinal reinforcement.** At any section of a beam, each masonry unit through the beam depth shall contain longitudinal reinforcement.

The variation in the longitudinal reinforcement area between units at any section shall not be greater than 50 percent, except multiple 12 mm diameter (#4) bars shall not be greater than 100 percent of the minimum area of longitudinal reinforcement contained by any one unit, except where splices occur.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15 f'_{m} / f_y$.

2. **Transverse reinforcement.** Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180-degree hook, as defined in Section 9.8.2.2.4, and shall be single pieces. Within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the beam.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the beam. Minimum reinforcement ratio shall be 0.0015. The first transverse bar shall not be more than 100 mm (4 inches) from the face of the pier.

9.8.2.6.2.6 Members subjected to axial force and flexure. The requirements set forth in this subsection apply to piers proportioned to resist flexure in conjunction with axial loads.

1. *Longitudinal reinforcement.* A minimum of four longitudinal bars shall be provided at all sections of every pier.

Flexural reinforcement shall be distributed across the member depth. Variation in reinforcement area between reinforced cells shall not exceed 50 percent.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15 f'_m / f_y$.

Maximum bar diameter shall be one eighth nominal width of the pier.

2. *Transverse reinforcement.* Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook as defined in Section 9.8.2.2.4.

Within an end region extending one pier depth from the end of the beam, and at any region at which flexural yielding may occur during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the pier.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the pier. The minimum transverse reinforcement ratio shall be 0.0015.

3. *Lateral reinforcement.* Lateral reinforcement shall be provided to confine the grouted core when compressive strains due to axial and bending forces exceed 0.0015, corresponding to factored forces with R_w equal to 1.5. The unconfined portion of the cross section with strain exceeding 0.0015 shall be neglected in computing the nominal strength of the section.

The total cross-sectional area of rectangular tie reinforcement for the confined core shall not be less than:

$$A_{sh} = 0.09 s h_c f'_m / f_{yh} \quad (9.8-40)$$

Alternatively, equivalent confinement which can develop an ultimate compressive strain of at least 0.006 may be substituted for rectangular tie reinforcement.

9.8.2.6.2.7 Pier design forces. Pier nominal moment strength shall not be less than 1.6 times the pier moment corresponding to the development of beam plastic hinges, except at the foundation level. Pier axial load based on the development of beam plastic hinges in accordance with the paragraph above and including factored dead and live loads shall not exceed $0.15 A_n f'_m$. The drift ratio of piers shall satisfy the limits specified in Chapter 5.

The effects of cracking on member stiffness shall be considered. The base plastic hinge of the pier must form immediately adjacent to the level of lateral support provided at the base or foundation.

9.8.2.6.2.8 Shear design

1. *General.* Beam and pier nominal shear strength shall not be less than 1.4 times the shears corresponding to the development of beam flexural yielding. It shall be assumed in the calculation of member shear force that moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span.

2. *Vertical member shear strength.* The nominal shear strength shall be determined from Formula (9.8-41):

$$V_n = V_m + V_s \quad (9.8-41)$$

Where:

$$V_m = 0.083C_d A_{mv} \sqrt{f'_m} \quad (9.8-42)$$

For FPS:

$$V_m = C_d A_{mv} \sqrt{f'_m}$$

and

$$V_s = A_{mv} \rho_n f_y \quad (9.8-43)$$

The value of V_m shall be zero within an end region extending one pier depth from beam faces and at any region where pier flexural yielding may occur during seismic loading, and at piers subjected to net tension factored loads. The nominal pier shear strength V_n , shall not exceed the value determined from Table 9.12.

3. *Beam shear strength.* The nominal shear strength shall be determined from Formula (9.8-44),

Where:

$$V_m = 0.01A_{mv} \sqrt{f'_m} \quad (9.8-44)$$

For FPS:

$$V_m = 1.2A_{mv} \sqrt{f'_m}$$

The value of V_m shall be zero within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic loading.

The nominal beam shear strength V_n , shall be determined from Formula (9.8-45).

$$V_n \leq 0.33A_{mv} \sqrt{f'_m} \quad (9.8-45)$$

For FPS:

$$V_n \leq 4A_{mv} \sqrt{f'_m}$$

9.8.2.6.2.9 Joints

1. *General requirements.* Where reinforcing bars extend through a joint, the joint dimensions shall be proportioned such that

$$h_p > 400 \frac{d_{bb}}{\sqrt{f'_g}} \quad (9.8-46)$$

For FPS:

$$h_p > 4800 \frac{d_{bb}}{\sqrt{f'_g}} \quad \text{and}$$

$$h_b > 150 \frac{d_{bp}}{\sqrt{f'_g}} \quad (9.8-47)$$

For FPS:

$$h_b > 1800 \frac{d_{bp}}{\sqrt{f'_g}}$$

The grout strength shall not exceed 35 MPa (5,000 psi) for the purposes of Formulas (9.8-46) and (9.8-47). Joint shear forces shall be calculated on the assumption that the stress in all flexural tension reinforcement of the beams at the pier faces is $1.4 f_y$. Strength of joint shall be governed by the appropriate strength reduction factors specified in Section 9.8.1.4.4.

Beam longitudinal reinforcement terminating in a pier shall be extended to the far face of the pier and anchored by a standard 90- or 180-degree hook, as defined in Section 9.8.2.2.4, bent back to the beam. Pier longitudinal reinforcement terminating in a beam shall be extended to the far face of the beam and anchored by a standard 90- or 180-degree hook, as defined in Section 9.8.2.2.4, bent back to the beam.

2. *Transverse reinforcement.* Special horizontal joint shear reinforcement crossing a potential corner-to-corner diagonal joint shear crack, and anchored by standard hooks, as defined in Section 9.8.2.2.4, around the extreme pier reinforcing bars shall be provided such that

$$A_{jh} = 0.5 V_{jh} / f_y \quad (9.8-48)$$

Vertical shear forces may be considered to be carried by a combination of masonry shear-resisting mechanisms and truss mechanisms involving intermediate pier reinforcing bars.

3. *Shear strength.* The nominal horizontal shear strength of the joint shall not exceed $0.58\sqrt{f'_m}$ (**For FPS:** $7\sqrt{f'_m}$) or 2.4 MPa (350 psi), whichever is less.

9.9 Empirical Design

9.9.1 Scope

Simple residential buildings having no vertical or horizontal irregularities as defined in Table 5.11 and 5.12 may be designed according to provisions of this section provided that:

9.9.1.1 The plan shape is rectangular having the length of the building not exceeding four times its width. In cases where longer building is required, provide seismic isolation joint to maintain specified length to width ratio.

9.9.1.2 The walls of the building shall be continuous from the foundations to the top of the building with the inter-storey height not exceeding 3m (10 ft).

9.9.1.3 A minimum of two parallel shear walls should be placed in each orthogonal direction. The walls to be categorized as shear walls shall have a minimum thickness of 200 mm (8"). The

height to thickness ratio (h/t) and length to height ratio l/h shall conform to values given in Table 9.14.

9.9.1.4 At least 75 % of the vertical loads should be supported by the shear walls.

9.9.1.5 In each of the two directions, shear walls should be placed with a spacing not exceeding:

- i. 6.75 m (22 ft) for masonry buildings with seismic bands
- ii. 7.5 m (25 ft) for Confined Masonry buildings
- iii. 9.0 m (30 ft) for Reinforced masonry buildings

9.9.1.6 For each storey, the ratio of the total cross-sectional area of shear walls in horizontal plane of the walls to the total floor area, should not be less than the values indicated in the Table 9.15 for each of the two orthogonal directions.

9.9.1.7 For each storey it should be verified that:

$$\sigma = N / A \leq 0.15 f'_{m}$$

where:

N is the total vertical load at the base of the considered wall.

A is the total area of the load-bearing walls (for vertical loads) of the same floor.

f'_{m} is the compressive strength in vertical direction of the masonry.

9.9.1.8 Masonry bearing walls built in mortar, as specified in 9.9.2.2 unless rationally designed as reinforced masonry shall not be built of greater height than 15 m (50 ft.) subject to a maximum of four storeys when measured from the mean ground level to the roof slab or ridge level. The masonry bearing walls shall be strengthened to ensure seismic resistance in accordance with section 9.9.3, 9.9.4 or 9.9.5.

9.9.1.9 The bearing walls in both directions shall be straight and symmetrical in plan as far as possible.

9.9.1.10 *Openings in the Bearing Walls*

- a) Door and window openings shall be as per guidelines given in Table 9.16 and Fig. 9.1.
- b) Openings in any storey shall have their top at the same level so that a continuous Band could be provided over them, including the lintels throughout the building.
- c) Where openings do not comply with the guidelines of Table 9.16, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, as shown in Fig. 9.2 with Grade 60 deformed steel bars of 8mm diameter (# 3) and the quantity in accordance with 9.9.3.10.
- d) If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.
- e) If an opening is tall from bottom to almost top of a storey, thus dividing the wall into two portions, these portions shall be reinforced with horizontal reinforcement of at least 6mm diameter bars at not more than 450 mm (18 in) intervals, one on inner and one on outer face, properly tied to vertical steel at jambs, corners or junction of walls, where used.

- f) The use of arches to span over the openings is a source of weakness and shall be avoided. Otherwise, steel or reinforced concrete ties should be provided.

9.9.2 Materials

9.9.2.1 Masonry units

The compressive strength of masonry units shall not be less than 8.25 MPa (1200 psi) for solid clay bricks and 11.75 MPa (1700 psi) for solid/hollow concrete blocks.

9.9.2.2 Mortar

The compressive strength of mortar shall not be less than 5.5 MPa (800 psi) and not greater than 75% of the compressive strength of masonry unit. The material strengths as specified in sections 9.9.2.1 and 9.9.2.2 may be relaxed by 20% in Seismic Zone 1.

9.9.2.3 Where steel reinforcing bars are provided in masonry, the bars shall be embedded in cement sand mortar with minimum clear cover of 15 mm (0.6 in) or bar diameter whichever more.

9.9.3 Masonry with Seismic Bands

9.9.3.1 Masonry with seismic bands shall be designed by the methods, as specified and detailed in subsequent clauses. Figs. 9.3 and 9.4 show, schematically, the overall strengthening arrangements to be adopted for Seismic Zones 3 and 4 comprising horizontal bands of reinforcement at critical levels and vertical reinforcing bars at corners, junctions of walls and jambs of opening. The strengthening requirements are listed in Table 9.17.

9.9.3.2 Lintel band is a band provided at lintel level on all load bearing internal, external longitudinal and cross walls. The specifications of the band are given in 9.9.3.5.

Note: Lintel band if provided in panel or partition walls also will improve their stability during severe earthquake

9.9.3.3 Roof band is a band provided immediately below the roof or floors. The specifications of the band are given in 9.9.3.5. Such a band need not be provided underneath reinforced concrete or brick-work slabs resting on bearing walls, provided that the slabs are continuous over the intermediate wall up to the expansion joints, if any, and cover the width of end walls, fully or at least $\frac{3}{4}$ of the wall thickness.

9.9.3.4 Gable band is a band provided at the top of gable masonry below the purlins. The specifications of the band are given in 9.9.3.5. This band shall be made continuous with the roof band at the eaves level.

9.9.3.5 Section and Reinforcement of Band

The band shall be made of reinforced concrete having a minimum compressive cylinder strength of 17 MPa (2500 psi) at 28 days. The bands shall be of the full width of the wall, not less than 75 mm (3 in.) in depth and reinforced with steel, as indicated in Table 9.18.

If the height of the gable exceeds 1200 mm (4 ft), vertical concrete element/elements shall be provided with a minimum width of 150 mm (6 in). One element at the center and others symmetrically placed on both sides of the central element spaced not more than 8 ft shall be provided. The element shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or $260 \text{ mm}^2 (0.40 \text{ in}^2)$, whichever is greater. The stirrups shall be at least 6.4 mm ($\frac{1}{4}$ in) in diameter and spaced at not more than 150 mm (6 in).

Note: a) In coastal areas, the concrete shall have a minimum compressive cylinder strength of 21 MPa (3000 psi) at 28 days.

b) For full integrity of walls at corners and junctions and effective horizontal bending resistance of bands, continuity of reinforcement is essential. The details as shown in Fig. 9.5 are recommended.

9.9.3.6 Plinth band is a band provided at plinth level on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties, as frequently happens in hill areas. Where used, its section may be kept same as in 9.9.3.5.

9.9.3.7 In seismic Zones 3 and 4, steel dowel bars shall be used at corners and T-junctions of walls at the sill level of windows for a length of 900 mm (3 ft) from the inside corner in each wall. Such dowels may be in the form of U stirrups of 8 mm diameter (# 3). Where used, such bars shall have a minimum cover of 10 mm (0.4 in.) on all sides to minimize corrosion.

9.9.3.8 Vertical Reinforcement

Vertical steel at corners and junctions of walls, which are up to 340 mm (1.5 brick) thick, shall be provided as specified in Table 9.19. For walls thicker than 340 mm, the area of the bars shall be proportionately increased. No vertical steel need be provided in Seismic Zones 1 and 2A.

9.9.3.9 The vertical reinforcement shall be properly embedded in foundations and roof slab or roof band so as to develop its tensile strength in bond. It shall be passing through the lintel bands and floor level bands in all storeys. Bars in different storeys shall suitably lapped.

Note: Typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 9.6.

9.9.3.10 Vertical reinforcement at jambs of window and door openings shall be provided as per Table 9.19. It may start from foundation and terminate in lintel band, in case of window openings with sill bands, the reinforcement may start from sill level.

9.9.4 *Confined Masonry*

9.9.4.1 For buildings not having reinforced concrete floors/roof, horizontal reinforced concrete beams shall be placed over all walls at every floor level. These beams shall form continuous bonding elements physically connected to each other.

9.9.4.2 The beam mentioned in section 9.9.4.1 shall have a minimum depth of 225 mm (9 in) and shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 515 mm^2 (0.8 in^2), whichever is greater. The stirrups shall be at least 6 mm ($\frac{1}{4}$ in) in diameter and spaced at not more than 115 mm ($4\frac{1}{2}$ in).

9.9.4.3 All masonry walls of the building shall be connected at the lintel level by a continuous reinforced concrete beam. The beam shall have a minimum depth of 150 mm (6 in) and shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 258 mm^2 (0.40 in^2), whichever is greater. The stirrups shall be at least 6 mm ($\frac{1}{4}$ in) in diameter and spaced at not more than 150 mm (6 in). Such beams need not to be provided in buildings located in Seismic Zone 1. However, the minimum bearing length of lintels shall be at least 225 mm (9 in) on each end.

9.9.4.4 Concrete beam shall also be provided over the gable walls with a minimum depth of 225 mm (9 in). The beam shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 515 mm^2 (0.8 in^2), whichever is greater. The stirrups shall be at least 6 mm ($\frac{1}{4}$ in) in diameter and spaced at not more than 150 mm (6 in).

9.9.4.5 If the height of the gable exceeds 1200 mm (4 ft), vertical concrete element/elements shall be provided with a minimum width of 150 mm (6 in). One element at the center and others symmetrically placed on both sides of the central element spaced not more than 8 ft shall be provided. The element shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 258 mm^2 (0.40 in^2) whichever is greater. The stirrups shall be at least 6 mm ($\frac{1}{4}$ in) in diameter and spaced at not more than 150 mm (6 in).

9.9.4.6 Masonry walls shall be tied to the vertical confining elements (refer 9.9.4.9) by providing horizontal reinforcement two 6 mm ($\frac{1}{4}$ in) diameter bars spaced at a maximum spacing of 600 mm (24 in). These bars should be properly anchored with ties of vertical confining elements and shall also be extended in to the wall up to a distance of 600 mm (2 ft).

9.9.4.7 In order to obtain an effective bond between the confining elements and the masonry, the concrete of the confining elements shall be cast after the masonry has been built with proper toothing in place.

9.9.4.8 The cross-sectional dimensions of both horizontal and vertical confining elements shall not be less than 150 mm (6 in).

9.9.4.9 Vertical confining elements shall be placed:

- i. At the free edges of each structural wall element;
- ii. At both sides of any wall opening with an area of more than:
 - a) 1.40 m^2 (15 ft^2) for Seismic Zones 3 and 4,
 - b) 1.86 m^2 (20 ft^2) for Seismic Zone 2, and
 - c) 2.32 m^2 (25 ft^2) for Seismic Zone 1.
- iii. Within the wall if necessary in order not to exceed a spacing of 4.5 m (15 ft) between the confining elements;
- iv. At the intersections of structural walls, wherever the confining elements imposed by the above provisions are at a distance larger than 1.5 m (5 ft) for Seismic Zones 2, 3 and 4 and 2 m (7 ft) for Seismic Zone 1.

9.9.4.10 Horizontal confining elements shall be placed in the plane of the wall at every floor level and in any case with a vertical spacing of not more than 3 m (10 ft).

9.9.4.11 The longitudinal reinforcement of vertical confining elements shall not have a cross-sectional area less than 258 mm^2 (0.4 in^2), nor more than 1% of the cross-sectional area of the confining element. Stirrups not less than 6 mm ($\frac{1}{4}$ in) in diameter and spaced not more than 150 mm (6 in) shall be provided around the longitudinal reinforcement.

9.9.4.12 Lap splices shall not be less than 60 bar diameters in length.

9.9.5 *Reinforced Masonry*

In addition to requirements of 9.9.4.1 to 9.9.4.4, the reinforced masonry shall also comply with the following requirements.

9.9.5.1 Horizontal reinforcement shall be placed in the bed joints or in suitable grooves in the units, with a vertical spacing not exceeding 600 mm (24 in).

9.9.5.2 Vertical reinforcement shall be located in pockets, cavities or holes in the masonry walls or units with a horizontal spacing not exceeding two times the thickness of wall nor 900 mm (36 in).

9.9.5.3 The horizontal and vertical reinforcement spread in the walls, as a percentage of the gross area of the section of the wall taken perpendicular to the reinforcement shall be at least 0.12% and 0.08% respectively.

9.9.5.4 Vertical reinforcements with a cross-sectional area of not less than 129 mm² (0.2 in²) shall be arranged:

- i. Within 300 mm (12 in) from the free edge of the wall element;
- ii. At every wall intersection;
- iii. Within 305 mm (12 in) from free edge on all sides of the openings having smaller dimension greater than 457 mm (18 in).

9.9.5.5 Lap splices shall not be less than 60 bar diameters in length.

Tables 9.1 – Mortar Proportions for Unit Masonry

Mortar	Type	Proportions by Volume (Cementitious Materials)						Aggregate Measured in a Damp, Loose Condition	
		Portland Cement or Blended Cement	Masonry Cement ¹			Mortar Cement ²			
			M	S	N	M	S		
Cement-lime	M	1	-	-	-	-	-	$\frac{1}{4}$	
	S	1	-	-	-	-	-	Over $\frac{1}{4}$ to $\frac{1}{2}$	
	N	1	-	-	-	-	-	Over $\frac{1}{2}$ to $\frac{1}{4}$	
	O	1	-	-	-	-	-	Over $1\frac{1}{4}$ to $2\frac{1}{2}$	
Mortar Cement	M	1	-	-	-	-	1	-	
	M	-	-	-	-	1	-	-	
	S	$\frac{1}{2}$	-	-	-	-	1	-	
	S	-	-	-	-	1	-	-	
	N	-	-	-	-	-	1	-	
Masonry Cement	M	1	-	-	1	-	-	-	
	M	-	1	-	-	-	-	-	
	S	$\frac{1}{2}$	-	-	1	-	-	-	
	S	-	-	1	-	-	-	-	
	N	-	-	-	1	-	-	-	
	O	-	-	-	1	-	-	-	

¹Masonry cement conforming to the requirements of UBC Standard 21-11.²Mortar cement conforming to the requirements of UBC Standard 21-14.**Table 9.2 – Grout Proportions by Volume¹**

Type	Parts by Volume of Portland Cement or Blended Cement	Parts by Volume of Hydrated Lime or Lime Putty	Aggregate Measured in a Damp, Loose Condition	
			Fine	Coarse
Fine grout	1	0 to 1/10	2 $\frac{1}{4}$ to 3 times the sum of the volumes of the cementitious materials	
Coarse grout	1	0 to 1/10	2 $\frac{1}{4}$ to 3 times the sum of the volumes of the cementitious materials	1 to 2 time the sum of the volumes of the cementitious materials

¹Grout shall attain a minimum compressive strength at 28 days of 2,000 psi (13.8 MPa). The building official may require a compressive field strength test of grout made in accordance with UBC standard 9.18.**Table 9.3 – Grouting Limitations**

Grout Type	Grout Pour Maximum Height (feet) ¹	Minimum Dimensions of the Total Clear Areas within Grout Spaces and Cells ^{2,3} (inches)	
		x25 for mm	
	x304.8 for mm	Multiwythe Masonry	Hollow-unit Masonry
Fine	1	$\frac{3}{4}$	$1\frac{1}{2} \times 2$
Fine	5	$1\frac{1}{2}$	$1\frac{1}{2} \times 2$
Fine	8	$1\frac{1}{2}$	$1\frac{1}{2} \times 3$
Fine	12	$1\frac{1}{2}$	$1\frac{3}{4} \times 3$
Fine	24	2	3×3
Coarse	1	$1\frac{1}{2}$	$1\frac{1}{2} \times 3$
Coarse	5	2	$2\frac{1}{2} \times 3$
Coarse	8	2	3×3
Coarse	12	$2\frac{1}{2}$	3×3
Coarse	24	3	3×4

¹See also Section 9.4.11.²The actual grout space or grout cell dimensions must be larger than the sum of the following items: (1) The required minimum dimensions of total clear areas in Table 9.3 (2) The width of any mortar projections within the space; and (3) The horizontal projections of the diameters of the horizontal reinforcing bars within a cross section of the grout space or cell.³The minimum dimensions of total clear areas shall be made up of one or more open areas, with at least one area being 20 mm ($\frac{3}{4}$ inch) or greater in width.

Table 9.4 – Specified Compressive Strength of Masonry, f'_m (MPa) Based on Specifying the Compressive Strength of Masonry Units

Compressive Strength of Clay Masonry Units ^{1,2} (MPa)	Specified Compressive Strength of Masonry, f'_m	
	Type M or S Mortar ³ (MPa)	Type N Mortar ³ (MPa)
x 145 for psi		
96 or more	36	30
83	32	26
69	28	23
55	23	19
41	19	15
28	14	11
Compressive Strength of Concrete Masonry Units ^{2,4} (psi)	Specified Compressive Strength of Masonry, f'_m	
	Type M or S Mortar ³ (psi)	Type N Mortar ³ (psi)
x 6.89 for kPa		
33 or more	21	19
26	17	16
19	14	13
13	10	9
9	7	7

¹Compressive strength of solid clay masonry units is based on gross area. Compressive strength of hollow clay masonry units is based on minimum net area. Values may be interpolated. When hollow clay masonry units are grouted, the grout shall conform to the proportions in Table 9.2.

²Assumed assemblage. The specified compressive strength of masonry f'_m is based on gross area strength when using solid units or solid grouted masonry and net area strength when using ungrouted hollow units.

³Mortar for unit masonry, proportion specification, as specified in Table 9.1. These values apply to Portland cement-lime mortars without added air-entraining materials.

⁴Values may be interpolated. In grouted concrete masonry, the compressive strength of grout shall be equal to or greater than the compressive strength of the concrete masonry units.

Table 9.5-1 – Allowable Tension, B_t , for Embedded Anchor Bolts for Clay and Concrete Masonry, Newton^{1,2,3}

f'_m (kPa)	Embedment Length l_b , or Edge Distance, l_{be} (mm))						
	50	75	100	125	150	200	250
	x 0.145 for psi	x 0.04 for inches x 0.225 for pounds					
10,335	1,065	2,445	4,315	6,760	9,745	17,310	27,055
12,400	1,200	2,670	4,760	7,430	10,680	18,955	29,635
13,780	1,250	2,800	4,980	7,830	11,210	20,025	31,240
17,225	1,380	3,160	5,605	8,720	12,590	22,380	34,930
20,670	1,515	3,425	6,140	9,565	13,795	24,520	38,270
27,560	1,780	3,960	7,075	11,035	15,930	28,300	44,185
34,450	1,955	4,450	7,920	12,370	17,800	31,640	49,395
41,430	2,135	4,850	8,675	13,525	19,490	34,665	54,290

¹The allowable tension values in Table 9.5-1 are based on compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension in pounds is given in Table 9.5-2.

²Values are for bolts of at least A 307 quality. Bolts shall be those specified in Section 9.6.2.14.1.

³Values shown are for work with or without special inspection.

Table 9.5-2 – Allowable Tension, B_t , for Embedded Anchor Bolts for Clay and Concrete Masonry, Newtons^{1,2}

Anchor Bolt Diameter (mm)							
x 0.04 for inches							
6	10	12	16	20	22	25	28
x 0.225 for pounds							
1,555	3,515	6,275	9,835	14,150	19,265	25,140	31,860

¹Values are for bolts of at least A 307 quality. Bolts shall be those specified in Section 9.6.2.14.1.

²Values shown are for work with or without special inspection.

Table 9.6 – Allowable shear, B_v , for Embedded Anchor Bolts for Clay and Concrete Masonry, Newton^{1,2}

f'_m (kPa)	Anchor Bolt Diameter (mm)						
	x 0.04 for inches						
	10	12	16	20	22	25	28
x 0.225 for pounds							
10,335	2,135	3,780	5,915	7,920	8,540	9,120	9,655
12,400	2,135	3,780	5,915	8,275	8,940	9,565	10,145
13,780	2,135	3,780	5,915	8,455	9,165	9,790	10,410
17,225	2,135	3,780	5,915	8,455	9,700	10,365	10,990
20,670	2,135	3,780	5,915	8,455	10,145	10,855	11,525
27,560	2,135	3,780	5,915	8,455	10,900	11,660	12,370
34,450	2,135	3,780	5,915	8,455	11,525	12,325	13,080
41,430	2,135	3,780	5,915	8,455	11,570	12,905	13,705

¹Value are for bolts of at least A 307 quality. Bolts shall be those specified in Section 9.6.2.14.1.²Values shown are for work with or without special inspection.**Table 9.7 – Minimum Diameters of Bend**

Bar Size	Minimum Diameter
No. 3 through No. 8	6 bar diameters
No. 9 through no. 11	8 bar diameters

Table 9.8 – Radius of Gyration¹ for Concrete Masonry Units²

Grout Spacing (mm)	Nominal Width of Wall (mm)				
	x 0.04 for inches				
x 0.04 for inches	100	150	200	250	300
Solid grouted	26	40	55	69	84
400	29	45	61	76	92
600	30	47	63	79	96
800	31	48	65	81	98
1000	31	48	66	83	99
1200	32	49	67	83	101
1400	32	49	67	84	101
1600	32	50	68	85	102
1800	33	50	68	85	103
No grout	33	52	71	89	107

¹For single-wythe masonry or for an individual wythe of a cavity wall.²The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow concrete masonry unit face shells and webs in accordance with UBC Standard 21-4 for two cell units.**Table 9.9 – Radius of Gyration¹ for Clay Masonry Unit Length, 400 mm²**

Grout Spacing (mm)	Nominal Width of Wall (mm)				
	x 0.04 for inches				
x 0.04 for inches	100	150	200	250	300
Solid grouted	26.5	41	56	70	85
400	29	45	61	76	91
600	30	46	63	78	94
800	31	47	64	80	96
1000	31	48	65	81	98
1200	32	48	65	82	98
1400	32	49	66	82	99
1600	32	49	66	83	99
1800	32	49	66	83	100
No grout	33	51	69	86	103

¹For single-wythe masonry or for an individual wythe of a cavity wall.

$$r = \sqrt{I/A_e}$$

²The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow concrete masonry face shells and webs in accordance with UBC Standard 21-1 for two cell units.

Table 9.10 – Radius of Gyration¹ for Clay Masonry Unit Length, 300 mm²

Grout Spacing (mm)	Nominal Width of Wall (mm)				
	x 0.04 for inches				
x 0.04 for inches	100	150	200	250	300
Solid grouted	27	41	56	71	85
300	29	44	60	75	90
450	30	46	62	77	93
600	30	46	63	78	94
750	31	47	63	79	95
900	31	47	64	79	96
1050	31	47	64	80	96
1200	31	48	64	80	96
1350	31	48	65	80	97
1500	32	48	65	80	97
1650	32	48	65	81	97
1800	32	48	65	81	97
No grout	32	49	66	82	99

¹For single-wythe masonry or for an individual wythe of a cavity wall.

$$r = \sqrt{I/A_e}$$

²The radius of gyration shall be based on the specified dimensions of the masonry units or shall be in accordance with the values shown which are based on the minimum dimensions of hollow concrete masonry face shells and webs in accordance with UBC Standard 21-1 for two cell units.

Table 9.11 – Allowable Flexural Tension (kPa)

Unit Type	Mortar Type			
	Cement-lime and Mortar Cement		Masonry Cement	
	M OR S	N	M OR S	N
	x 0.145 for kPa			
Normal to bed joints				
Solid	276	207	165	103
Hollow	172	131	103	62
Normal to head joints				
Solid	550	413	331	207
Hollow	345	262	207	124

Table 9.12 – Maximum Nominal Shear Strength Values^{1,2}

M/Vd^l	C_d
≤ 0.25	$322A_e\sqrt{f'_m} \leq 1691A_e$ ($6.0A_e\sqrt{f'_m} \leq 380A_e$)
≥ 1.00	$214A_e\sqrt{f'_m} \leq 1113A_e$ ($4.0A_e\sqrt{f'_m} \leq 250A_e$)

¹M is the maximum bending moment that occurs simultaneously with the shear load V at the section under consideration.

Interpolation may be by straight line for M/Vd values between 0.25 and 1.00.

²V_n is in N (lb) and f'_m is in kPa (psi).

Table 9.13 – Nominal Shear Strength Coefficient

M/Vd^l	C_d
≤ 0.25	2.4
≥ 1.00	1.2

¹M is the maximum bending moment that occurs simultaneously with the shear load V at the section under consideration.

Interpolation may be by straight line for M/Vd values between 0.25 and 1.00.

Table 9.14 – Geometric Requirements for Shear Walls

Type of Masonry	$(h/t)_{\max}$	$(l/h)_{\min}^*$
Masonry with seismic bands	12	0.4
Confined masonry	15	0.3**
Reinforced masonry	15	No restriction

*. $(l/h)_{\min}$ may be reduced by 20% for Seismic Zone 1

** Opening can be provided at the end of wall provided vertical confining element is provided on both sides of opening.

Table 9.15 - Minimum Area of the Resistant Walls in term of Percentage of Floor Area in each Orthogonal Direction for Simple Buildings

Type of Structure	Number of Storeys	Seismic Zones				
		Zone 1	Zone 2A	Zone 2B	Zone 3	Zone 4
Masonry with seismic bands	1	3.50 %	4.00%	4.00 %	5.50%	6.00%
	2	4.00 %	4.50%	5.00%	6.00%	
	3	4.5 %	5.00%	5.50%		
Confined masonry	1	3.00%	3.50%	3.50%	4.00%	5.00%
	2	4.00%	4.00%	4.00%	4.50%	6.00%
	3	4.5 %	4.5 %	5.00%	5.50%	
Reinforced masonry	1	3.00 %	3.00%	3.00 %	3.50 %	4.50 %
	2	3.50 %	3.50%	3.50 %	4.00 %	5.00 %
	3	4.00%	4.00%	4.50%	5.00 %	

Table 9.16 - Size and Position of Openings in Bearing Walls (Refer Fig. 9.1)

Sr. No. (1)	Position of Opening (2)	Details of Opening		
		Zone 1 & 2A (3)	Zone 2B (4)	Zone 3 & 4 (5)
i)	Distance b_s from the inside corner of outside wall, Min.	0	230 mm	450 mm
ii)	For total length of openings, the ratio $(b_1+b_2+b_3)/l_1$ or $(b_6+b_7)/l_2$ shall not exceed:			
	a) one-storeyed building	0.60	0.55	0.50
	b) two-storeyed building	0.50	0.46	0.42
	c) three or four-storeyed building	0.42	0.37	0.33
iii)	Pier width between consecutive openings b_4 , Min	340 mm	450 mm	560 mm
iv)	Vertical distance between two openings one above the other h_3 , Min	600 mm	600 mm	600 mm
v)	Width of opening of ventilator b_s , Max	900 mm	900 mm	900 mm

**Table 9.17 - Strengthening Arrangements Recommended for Masonry Buildings
(Rectangular Masonry Units)**

Seismic Zone (1)	Number of Storeys (2)	Strengthening to be Provided in all Storeys (3)
1 & 2A	i) 1 to 3 ii) 4	a
2B	i) 1 to 3 ii) 4	a, b, c a, b, c, f, g a, b c, d, f g
3	i) 1 and 2 ii) 3 and 4	a, c, d, f, g a to g
4	i) 1 and 2 ii) 3 and 4	a to g a to h

Where

- a – Masonry mortar as specified in 9.9.2.2.
- b – Lintel band
- c – Roof band and gable band where necessary
- d – Vertical steel at corners and junctions of walls
- e – Vertical steel at jambs of openings
- f – Bracing in plan at tie level of roofs
- g – Plinth band where necessary, and
- h – Dowel bars

NOTE – In case of four storey buildings of Seismic Zone 2B, the requirements of vertical steel may be checked through a seismic analysis. If the analysis shows that vertical steel is not required, the designer may take the decision accordingly.

Table 9.18 - Recommended Longitudinal Steel in Reinforced Concrete Bands

Span m (1)	Seismic Zone 2B		Seismic Zone 3		Seismic Zone 4	
	No. of Bars (2)	Dia mm (3)	No. of Bars (4)	Dia mm (5)	No. of Bars (6)	Dia mm (7)
	5 or less	2	8	2	8	2
6	2	8	2	8	2	10
7	2	8	2	10	2	12
8	2	10	2	12	4	10

NOTES

- Span of wall shall be the distance between centre lines of its cross walls or buttresses. For spans greater than 8 m (26 ft.) insert pilasters or buttresses to reduce the span or special calculations shall be made to determine the strength of wall and section of band.
- The number and diameter of bars given above pertain to high strength deformed bars. If plain mild steel bars are used keeping the same number, the following diameters may be used:

High strength deformed steel bar diameter	8	10	12	16	20
Mild steel plain deformed bar diameter	10	12	16	20	25

3 Width of RC band is assumed same as the thickness of the wall. Wall thickness shall be 200 mm (8 in.) minimum. A clear cover of 20 mm (0.75 in.) from face of wall will be maintained.

4 The vertical thickness of RC band be kept 75 mm (3 in.) minimum, where two longitudinal bars are specified, one on each face; and 150 mm, where four bars are specified.

5 The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm dia spaced at 150 mm (6 in.) apart.

Table 9.19 - Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

No. of Storeys (1)	Storey (2)	Diameter of Single Bar in mm at each Critical Section		
		Seismic Zone 2B (3)	Seismic Zone 3 (4)	Seismic Zone 4 (5)
One	-	Nil	Nil	10
Two	Top	Nil	Nil	10
	Bottom	Nil	Nil	
Three	Top	Nil	10	10
	Middle	Nil	10	12
	Bottom	Nil	12	12
Four	Top	10	10	10
	Third	10	10	12
	Second	10	12	16
	Bottom	12	12	20

NOTES

- 1 The diameters given above are for high strength deformed steel bars. For mild steel plain bars, use equivalent diameters as given in Table 9.18 (Note 2).
- 2 The vertical bars will be covered with concrete 1:2:4 or mortar 1:3 in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry.

CHAPTER 10

ARCHITECTURAL ELEMENTS

10.1 Symbols and Notations

b_p	=	Width of the rectangular glass panel.
C_c	=	Seismic coefficient for architectural components from Table 10.1.
c_1	=	Clearance (gap) between the vertical glass edges and the frame.
c_2	=	Clearance (gap) between the horizontal glass edges and the frame.
D_{clear}	=	Relative horizontal (drift) displacement.
D_p	=	Relative seismic displacement.
F_c	=	Seismic force applied to a component of a building at its center of gravity.
h_p	=	Height of the rectangular glass panel.
I	=	Occupancy Importance factor.
P	=	Performance criteria factor from Table 10.1.
W_c	=	Weight of the architectural component.
Z	=	Seismic zone factor from Table 5.9.
$\Delta_{fallout}$	=	Relative seismic displacement (drift) at which glass fallout.

10.2 Seismic Loads Applied to Architectural Components

Architectural components and their means of attachment shall be designed for seismic forces determined in accordance with the following:

$$F_c = ZC_c PW_c \quad (10.2-1)$$

The force shall be applied independently along all orthogonal directions in combination with the static loads of the component. Performance criteria factor P based on occupancy categories are defined in Table 10.1.

10.2.1 Component Force Application

The component seismic force shall be applied at the center of gravity of the component nonconcurrently in any horizontal direction.

10.2.2 Component Force Transfer

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

10.2.3 Architectural Component Deformation

Architectural components shall be designed for the design storey drift of the seismic force-resisting system as determined in Section 5.30.10 and shall also be designed for vertical deflection due to joint rotation of cantilever structural members with the exception that components having a performance criteria factor P of 0.5 are to be designed for 50 percent of the design storey drift.

The interrelationship of systems or components and their effect on each other shall be considered so that the failure of an architectural component shall not cause the failure of an architectural system or component with a higher performance criteria factor P . The effect on the

response of the structural system and deformational capability of architectural components shall be considered where these systems or components interact with the structural system.

10.2.4 *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 Equation 10.2-1 shall not exceed the deflection capability of the component or system.

10.3 *Suspended Ceilings*

Provision shall be made for the lateral support and/or interaction of other components that may be incorporated into the ceiling and may impose seismic forces into the ceiling system.

10.3.1 *Seismic Forces*

The weight of the ceiling W_c shall include the ceiling, grid and panels; light fixtures if attached to, clipped to, or laterally supported by the ceiling grid, and other components that are laterally supported by the ceiling. W_c shall be taken as not less than 0.19 kPa (4 psf).

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

10.3.2 *Integral Construction*

The sprinkler system and ceiling grid are permitted to be designed and tied together as an integral unit as an alternate to providing large clearances around sprinkler system penetrations through ceiling systems. Such a design shall consider the mass and flexibility of all elements involved, including the ceiling system, sprinkler system, light fixtures, mechanical, HVAC appurtenances etc. Such design shall be performed by a registered design professional.

10.3.3 *Access Floors*

10.3.3.1 *General*

The weight of the access floor W_c shall include the weight of the floor system, 100 percent of the weight of all equipment fastened to the floor and 25 percent of the weight of all equipment supported by, but not fastened to the floor. The seismic force F_c , shall be transmitted from the top surface of the access floor to the supporting structure. Overturning effects of equipment fastened to the access floor panels with pedestals also shall be considered. Where checking individual pedestals for overturning effects, the maximum concurrent axial load shall not exceed the portion of W_c assigned to the pedestal under consideration.

10.3.3.2 *Special access floors*

Access floors shall be considered to be “special access floors” if they are designed to comply with the following considerations:

1. Connections transmitting seismic loads consist of mechanical fasteners, anchors satisfying the requirements of Appendix D of ACI 318, welding, or bearing. Design load capacities comply with recognized design codes and/or certified test results.
2. Seismic loads are not transmitted by friction, power actuated fasteners, adhesives, or by friction produced solely by the effects of gravity.

3. The design analysis of the bracing system includes the destabilizing effects of individual members buckling in compression.
4. Bracing and pedestals are of structural or mechanical shapes produced to ASTM specifications that specify minimum mechanical properties. Electrical tubing shall not be used.
5. Floor stringers that are designed to carry axial seismic loads and that are mechanically fastened to the supporting pedestals are used.

10.4 Partitions

10.4.1 General

Partitions that are tied to the ceiling and all partitions greater than 1.8 m (6 ft) in height shall be laterally braced to the building structure. Such bracing shall be independent of any ceiling splay bracing. Bracing shall be spaced to limit horizontal deflection at the partition head

Exceptions: Partitions that meet all of the following conditions:

1. The partition height does not exceed 3 m (10 ft).
2. The linear weight of the partition does not exceed the product of 0.479 kPa (10 psf) times the height in m (ft) of the partition.
3. The partition horizontal seismic load does not exceed 0.24 kPa (5 psf).

10.4.2 Glass in Glazed Curtain Walls, Glazed Storefronts and Glazed Partitions

10.4.2.1 General

Glass in glazed curtain walls, glazed storefronts, and glazed partitions shall meet the relative displacement requirement of Eq. 10.4-1.

$$\Delta_{fallout} \geq 1.25ID_p \quad (10.4-1)$$

or 13 mm (1/2 in), whichever is greater

Where:

- $\Delta_{fallout}$ = Relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront wall, or partition occurs.
- D_p = Relative seismic displacement that the component must be designed to accommodate. D_p shall be applied over the height of the glass component under consideration.
- I = Occupancy Importance factor (Chapter 5, Table 5.10).

Exceptions:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame shall not occur at the design drift, as demonstrated by Eq. 10.4-2, need not comply with this requirement:

$$D_{clear} \geq 1.25D_p \quad (10.4-2)$$

where:

D_{clear} = Relative horizontal (drift) displacement, measured over the height of the glass panel under consideration, which causes initial glass-to-frame contact.

For rectangular glass panels within rectangular wall frame:

$$D_{clear} = 2c_1 \left(1 + \frac{h_p c_2}{b_p c_1} \right) \quad (10.4-3)$$

where:

- h_p = the height of the rectangular glass panel.
 b_p = the width of the rectangular glass panel.
 c_1 = the clearance (gap) between the vertical glass edges and the frame.
 c_2 = the clearance (gap) between the horizontal glass edges and the frame.

2. Fully tempered monolithic glass in Occupancy Categories 1, 2, and 3 located no more than 3 meter (10 feet) above a walking surface need not comply with this requirement.
3. Annealed or heart-strengthened laminated glass in single thickness with interlayer no less than 0.76 mm (1/32 in) that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of 13 mm (1/2 in) minimum glass contact width, or other approved anchorage system need not comply with this requirement.

10.4.2.2 Seismic drift limits for glass components

The drift causing glass fallout from the curtain wall, storefront, or partition shall be determined by a rational engineering analysis or in accordance with manufacturer's standards and specifications.

Table 10.1—Architectural Component Seismic Coefficient (C_c) and Performance Criteria Factor (P)^a

Architectural Component	Component Seismic Coefficient (C_c)	Performance Criteria Factor (P)		
		Occupancy Category		
		1	2 & 3	4 & 5
Exterior nonbearing wall	0.9	1.0 ^d	1.5 ^b	1.5
Interior nonbearing walls ^b				
Stair enclosures	1.5	1.5	1.0 ^c	1.5
Elevator shaft enclosures	1.5	0.5 ^e	0.5 ^c	1.5
Other vertical shaft enclosures	0.9	1.0	1.0	1.5
Other nonbearing walls and partitions	0.9	1.0	1.0	1.5
Cantilever elements parapets, chimneys or stacks	3.0	1.5	1.5	1.5
Wall attachments	3.0	1.0 ^d	1.5 ^b	1.5
Veneer connections	3.0	0.5	1.0 ^g	1.0
Penthouses	0.6	NR	1.0	1.0
Structural fireproofing	0.9	0.5 ^f	1.0 ^c	0.5
Ceilings				
Fire-rated membrane	0.9	1.0	1.0	1.5
Nonfire-rated membrane	0.6	0.5	1.0	1.0
Storage racks more than 2.5 m high(contents included) ^j	1.5	1.0	1.0	1.5
Access floors (supported equipment included)	0.9	0.5	1.0	1.5
Elevator and counterweight guide rails and supports	1.25	1.0	1.0	1.5
Appendages				
Roofing units	0.6	NR	1.0 ^b	1.0 ^b
Containers and miscellaneous components (free standing)	1.5	NR	1.0	1.0

NR = Not required.

^a Exceptions:

1. Architectural components in buildings assigned to Seismic Zone 1 are exempt from the requirements of Chapter 10.
2. Architectural components and systems in buildings assigned to Seismic Zones 2A & 2B and Occupancy Category 1 that have a Performance Criteria Factor of 0.5 are exempt from the requirements of Chapter 10.

^b P may be reduced by 0.5 if the area facing the exterior wall is normally inaccessible for a distance of 3 meter (10 feet) and the building is only one storey.

^c P shall be increased by 0.5 if the building is more than four storeys or 12 meter (40 feet) in height.

^d P shall be increased by 0.5 if the area facing the exterior wall is normally accessible within a distance of 3 meter (10 feet) plus 0.3 meter (1 foot) for each floor height.

^e P may be reduced to NR if the building is less than 12 meter (40 feet) in height.

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

^g P may be reduced by 0.5 if the area facing the exterior wall is normally inaccessible for a distance of 3 meter (10 feet) plus 0.3 meter (1 foot) of each floor of height.

^h Exterior and interior bearing walls and their anchorage shall be designed for a force normal to the surface equal to Z times the weight of wall W_c , with a minimum force of 10 percent of the weight of the wall.

ⁱ The contents included in W_c may be reduced to 50% of the rated capacity for steel storage rack systems arranged such that in each direction the lines of framing that are designed to resist lateral forces consist of at least four columns connected to act as braced frames or moment resisting frames.

^j Storage shelving under 2.5 meter (8 feet) high, shall be considered miscellaneous components.

CHAPTER 11

MECHANICAL AND ELECTRICAL SYSTEMS

11.1 Symbols and Notations

F_m	=	Seismic force applied to an equipment or its component at its center of gravity.
Z	=	Seismic zone factor from Table 5.9.
C_c	=	Seismic coefficient for mechanical and electrical equipment from Table 11.1.
P	=	Performance criteria factor from Table 11.1.
a_c	=	Amplification factor from Table 11.2.
W_m	=	Weight of the equipment or its components.
T	=	Fundamental period of the building.
T_m	=	Component fundamental period.
g	=	Gravitational acceleration.
K_m	=	Stiffness of resilient support system of the component and attachment, determined in terms of load per unit deflection at the center of gravity of the component.

11.2 Seismic Loads Applied to Mechanical and Electrical Components

Mechanical and electrical components and their supports shall satisfy the requirements of this section. The design criteria for systems or components shall be included as part of the design documents.

An analysis of a component supporting mechanism based on established principles of structural dynamics may be performed to justify reducing the forces determined in this section. The attachment of mechanical and electrical components and their supports to the structure shall consider dynamic effects of the components, their contents, and their supports. Attachments for floor- or roof-mounted equipment weighing less than 2.0 kN and furniture need not be designed. Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces. Combined states of stress, such as tension and shear in anchor bolts, shall be investigated in accordance with established principles of mechanics.

When the structural failure of the lateral-force-resisting systems of equipment would cause a life hazard, such systems shall be designed to resist the following seismic forces:

$$F_m = ZC_c P a_c W_m \quad (11.2-1)$$

The force shall be applied independently in all orthogonal directions in combination with the static loads of the component. Performance criteria factor P shall be based on occupancy categories. Alternatively, the seismic forces are to be determined by a properly substantiated dynamic analysis subject to approval by the authority having jurisdiction.

11.2.1 Component Force Application

The component seismic force shall be applied at the center of gravity of the component non-concurrently in any horizontal direction. Mechanical and electrical components and systems shall be designed for an additional vertical force of 33 percent of the horizontal force acting up or down.

11.2.2 Component Force Transfer

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

11.2.3 Component Period

The fundamental period of the mechanical and electrical components (and their attachment to the building), T_m , shall be determined by the following equation provided that the component and attachment can be reasonably represented analytically by a simple spring and mass single degree-of-freedom system:

$$T_m = 2\pi \sqrt{\frac{W_m}{K_m g}} \quad (11.2-2)$$

Alternatively, the fundamental period of the component is permitted to be determined from experimental test data or by a properly substantiated analysis.

11.2.4 Component Attachment

The interrelationship of systems or components and their effect on each other shall be considered so that the failure of mechanical or electrical system or component shall not cause the failure of a mechanical or electrical system or component with a higher performance criteria factor P . The effect on the response of the structural system and deformational capability of electrical and mechanical systems or components shall be considered where these systems or components interact with the structural system.

11.2.5 Provision shall be made to eliminate seismic impact for components vulnerable to impact, for components constructed of nonductile materials, and in cases where material ductility will be reduced due to service conditions (e.g. low temperature applications).

11.2.6 The possibility of loads imposed on components by attached utility or service lines, due to differential movement of support points on separate structures, shall be evaluated.

11.2.7 Where piping or HVAC ductwork components are attached to structures that could displace relative to one another and for isolated structures where such components cross the isolation interface, the components shall be designed to accommodate the seismic relative displacements.

11.3 Elevator Design Requirements

The design and construction of elevators and components shall conform to the requirements of ANSI/ASME A17.1-1987, *American National Standard Safety Code for Elevators and Escalators*, including Appendix F, "Recommended Elevator Safety Requirements for Seismic Zone 2B, or greater."

Table 11.1 – Mechanical and Electrical Components and Systems Seismic Coefficient (C_c) and Performance Criteria Factor (P)^a

Mechanical and Electrical Component or System	Component Seismic Coefficient (C_c)^b	Performance Criteria Factor (P)		
		Occupancy Category		
		1	2 & 3	4 & 5
Fire protection equipment and systems	2.0	1.5	1.5	1.5
Emergency or stand-by electrical systems	2.0	1.5	1.5	1.5
Elevator drive, suspension system, and controller anchorage	1.25	1.0	1.0	1.5
General equipment				
Boilers, furnaces, incinerators, water heaters and other equipment using combustible energy sources or high-temperature sources, chimneys, flues, smoke stacks and vents				
Communication systems				
Electrical bus ducts, conduit and cable trays ^c	2.0	0.5	1.0	1.5
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				
Manufacturing and process machinery	0.67	0.5	1.0	1.5
Pipe systems				
Gas and high hazard piping	2.0	1.5	1.5	1.5
Fire suppression piping	2.0	1.5	1.5	1.5
Other pipe systems ^d	0.67	NR	1.0	1.5
HVAC and service ducts ^e	0.67	NR	1.0	1.5
Electrical panel boards and dimmers	0.67	NR	1.0	1.5
Lighting fixtures ^f	0.67	0.5	1.0	1.5
Conveyor systems (non personnel)	0.67	NR	NR	1.5

NR= Not required.

^aExceptions:

1. Mechanical and electrical components and systems in buildings assigned to Seismic Zones 2A & 2B and occupancy Category 1 that have a Performance Criteria Factor of 0.5 are exempt from the requirements of Chapter 11.
2. Mechanical and electrical components and systems in buildings assigned to Seismic Zone 1 are exempt from the requirements of Chapter 11.
3. Elevator components and system in buildings assigned to Seismic Zone 1 are exempt from requirement of Chapter 11.

^b C_c values are for horizontal forces; C_c values for vertical forces shall be taken as one-third of the horizontal values.^cSeismic restraints are not required for electrical conduit and cable trays for any of the following conditions: (1) conduit and cable trays suspended by individual hangers 300 mm (12 inches) or less in length from the top of the conduit to the supporting structure, (2) conduit in boiler and mechanical rooms that has less than 31 mm (1.25inch) inside diameter, (3) conduit in other areas that has less than 60 mm (2.5 inch) inside diameter.^dSeismic restraints are not required for any of the following conditions for other pipe systems: (1) piping suspended by individual hangers 300 mm (12 inches) or less in length from the top of the pipe to the supporting structure, (2) piping in boiler and mechanical rooms that has less than 30 mm (1.25 inch) inside diameter, (3) piping in other areas that has less 60 mm (2.5 inch) inside diameter.^eSeismic restraints are not required for any one of the following conditions for HVAC or service ducts: (1) ducts suspended by hangers 300 mm (12 inches) or less in length from the top of the duct to the supporting structure, (2) ducts that have a cross sectional area less than 0.55 square meter (6 square feet).^fPendulum lighting fixtures shall be designed using a Component Seismic Coefficient (C_c) of 1.5. The vertical support shall be designed with a factor of safety of 4.0.

Table 11.2 – Attachment Amplification Factor (a_c)

Component Supporting Mechanism	Attachment Amplification Factor (a_c)
Fixed or direct connection	1.0
Seismic-activated restraining device	1.0
Resilient support system where ^a :	
$T_m/T < 0.6$ or $T_m/T > 1.4$	1.0
$T_m/T \geq 0.6$ or $T_m/T \leq 1.4$	2.0

^a T is the fundamental period of the building in seconds determined by 5.30.2.2.

T_m is the fundamental period in seconds of the component and its means of attachment determined by 11.2.3.

APPENDIX A

BACKGROUND FOR SEISMIC ZONING MAP

A.1 Symbols and Notations

a	=	Co-efficient of magnitude-frequency curve
b	=	Co-efficient of magnitude-frequency curve
m	=	Magnitude
M_w	=	Moment Magnitude
M_L	=	Local Magnitude
m_b	=	Body-wave magnitude
M_s	=	Surface-wave magnitude
M_o	=	Seismic Moment
PGA	=	Peak Ground Acceleration
t	=	Time Period
v_s	=	Shear wave velocity

A.2 Overview

A.2.1 General

Being located close to the collision boundary of the Indian and Eurasian plates, Pakistan lies in a seismically active zone. Owing to high population density near seismically active areas, it is imperative that buildings should withstand the seismic hazard to which these may be exposed during their life time.

This appendix is based on a rigorous exercise based on compilation of geological, tectonic and seismicity data from Pakistan and its immediate surroundings. Only a brief account of salient seismotectonic features, seismicity and methodology adopted for seismic hazard zonation is included in this appendix. A separate document titled “Building Code of Pakistan, Seismic Hazard Evaluation Studies (2007)” contains details of geodynamics, tectonic zones, major faults, historical and instrumental seismicity catalogue and procedures adopted for seismic hazard evaluation.

A.2.2 Major Faults of Pakistan

Pakistan is characterized by extensive zones of moderate to high seismicity, induced by the regional collisional tectonics associated with Indian and Eurasian plates and resulting in manifestation of great Himalayan and associated mountain ranges. The geographic domain of Pakistan comprises a network of active seismotectonic features of regional extent, generally associated with collisional mountain ranges. These define four broad seismotectonic zones including 1) the Himalayan seismotectonic zone in the north, 2) Suleiman-Kirthar thrust-fold belt,

3) Chaman-Ornach Nal Transform Fault Zone, and 3) Makran Subduction Zone in the west, and 4) Rann of Kutch Seismotectonic Zone in the southeast. The Pamir-Hindukush Seismic Zone straddles across Afghanistan and Tajikistan outside Pakistan but in close vicinity of the NW Pakistan comprising District Chitral.

A great diversity of geological faults constitutes these seismotectonic zones. The most prominent fault types of the region include transform, wrench, thrust, and basement geo fractures. Major active faults of Pakistan and surrounding areas that strongly influence the seismic hazard are listed below:

Main Karakoram Thrust

Main Mantle Thrust

Raikot Fault

Main Boundary Thrust

Panjal-Khairabad Thrust

Himalayan Frontal Thrust

Riasi Thrust

Jhelum Fault

Salt Range Thrust

Kalabagh Fault

Bannu Fault

Kurram Fault

Chaman Transform Fault

Ornach-Nal Transform Fault

Quetta-Chiltan Fault

Kirthar Fault

Pab Fault

Kutch Mainland Fault

Allah Bund Fault

Nagar Parkar Fault

Hoshab Fault

Nai Rud Fault

Makran Coastal Fault

A.2.3 Seismicity

The information about earthquakes in this region is available in two forms i.e. historically recorded and instrumentally recorded earthquakes. The instrumentally recorded earthquake data is available only since 1904. Before this, the source of earthquake information is through the historical records and published literature which is also limited. The number of seismic stations remained small in South Asian region until 1960 when the installation of high quality seismograph under World Wide Standard Seismograph Network (WWSSN) increased the quality

of earthquake recording. In the present seismic studies, two classes of instrumental earthquake data have been used. The first one is based upon earthquake data from regional data catalogues compiled by International Seismological Centre (ISC) and National Earthquake Information Centre (NEIC) of USGS, and the other from earthquakes recorded by local networks of Pakistan Meteorological Department, Pakistan Atomic Energy Commission (PAEC) and Water & Power Development Authority (WAPDA).

A composite list of earthquakes recorded in and around Pakistan was prepared from the data collected from regional as well as local networks mentioned above. The duplicate earthquakes appearing in different catalogues were removed based on reported time and where time is not available based on the epicentral data. In preparation of this composite list preference was given to ISC catalogue as they have relocated and reviewed earthquake data after incorporating the available data from local stations also. The fault plane solutions available in the literature and provided by PAEC and Quaid-e-Azam University were also used to assess the source characteristics.

A.3 Seismic Hazard Evaluation Procedure

As per international practice and guidelines for seismic hazard evaluation and seismic hazard mapping for Building Codes, probabilistic seismic hazard assessment (PSHA) procedure was employed for seismic hazard analysis of Pakistan.

A.3.1 PSHA Methodology

In probabilistic hazard evaluation, the seismic activity of seismic source (line or area) is specified by a recurrence relationship, defining the cumulative number of events per year versus the magnitude. Distribution of earthquakes is assumed to be uniform within the source zone and independent of time.

The principle of the analysis, first developed by Cornell (1968) and later refined by various researchers, is to evaluate at the site of interest the probability of exceedance of a ground motion parameter (e.g. peak ground acceleration PGA) due to the occurrence of a strong event around the site. This approach combines the probability of exceedance of the earthquake size (recurrence relationship), and probability on the distance from the epicenter to the site.

Each seismic source zone is split into elementary zones at a certain distance from the site. Integration is carried out within each zone by summing the effects of the various elementary source zones taking into account the attenuation effect with distance. Total hazard is finally obtained by adding the influence of various sources. The results are expressed in terms of a ground motion parameter associated with return period (return period is the inverse of the annual frequency of exceedance of a given level of ground motion).

A seismic hazard model is developed based on findings of the seismotectonic synthesis. The seismic hazard model relies upon the concept of seismotectonic zones. Each seismic source zone is defined as a zone with homogenous seismic and tectonic features, inferred from geological, tectonic and seismic data. These zones are first defined, and then a maximum earthquake and an earthquake recurrence equation are elaborated for each of these seismic source zones.

The seismic parameters attached to the various seismic source zones are: a recurrence relationship relating the number of events for a specific period of time to the magnitude; the maximum earthquake giving an upper bound of potential magnitude in the zone; and an attenuation relationship representing the decrease of acceleration with distance.

A.3.2 Source Modeling – Area and Fault Seismic Sources

For the definition of seismic sources, either line (i.e. fault) or area sources can be used for modeling. Because of uncertainty in the epicenters location, it is not possible to relate the recorded earthquakes to the faults and to develop recurrence relationship for each fault and use them as exponential model. The whole area of Pakistan was therefore divided into seventeen area source zones (area sources) based on their homogeneous tectonic and seismic characteristics, keeping in view the geology, tectonics and seismicity of each area source zone. The eight area seismic source zones in the northern part of Pakistan are named Hindukush, Pamir, Kohistan, Hazara, Himalayas, Salt Range-Potwar, Bannu and Punjab seismic source zones. The nine area source zones in southern part of Pakistan are named Suleiman, Sibbi, Kirthar, Kurram-Chaman, Indus plateform, Rann of Kutch, Cholistan-Thar desert, Chagai and Makran.

Each of these area sources was assigned a maximum magnitude based on maximum recorded seismicity and a minimum magnitude based on threshold magnitude observed in the magnitude-frequency curve for the zone. As the shallow earthquakes are of more concern to seismic hazard, the minimum depth of the earthquakes is taken as 5-10 km for all sources except for Punjab seismic source zone where the minimum depth of earthquakes is taken as 20 km and for Hindukush zone where minimum depth was taken as 70 km.

The area source zones do not completely account for the long term seismicity associated with major active faults. In order to account for seismicity with large return period, the major active faults of Pakistan listed in Section A.2.2 were also modeled as characteristic fault sources for PSHA.

A.3.3 Earthquake Recurrence Model

A general equation that described earthquake recurrence may be expressed as follows:

$$N(m) = f(m, t) \quad (1)$$

Where $N(m)$ is the number of earthquakes with magnitude equal to or greater than m , and t is time period.

The simplest form of equation (1) that has been used in most engineering applications is the well known Richter's law which states that the cumulated number of earthquakes occurred in a given period of time can be approximated by the relationship

$$\log N(m) = a - b m \quad (2)$$

Equation (2) assumes spatial and temporal independence of all earthquakes, i.e. it has the properties of a Poisson model. Coefficients 'a' and 'b' can be derived from seismic data related to the source of interest. Coefficient 'a' is related to the total number of events occurred in the source zone and depends on its area, while coefficient 'b' represents the coefficient of proportionality between $\log N(m)$ and the magnitude.

The composite list of earthquakes prepared for areas in and around Pakistan provided the necessary database for the computation of b -value for each area source zone.

The updated composite seismic data from 1904-2006 contain magnitude values in the form of surface wave, body wave, local or duration magnitude type. Since each attenuation relationship is based on magnitude of given type, a single type must be selected. For data to be used in seismic

hazard analysis, all the magnitudes were therefore converted to moment magnitude (M_w) by the following equations.

Conversion from M_S and m_b to M_w was achieved through latest equation suggested by Scordilis (2006):

$$\begin{aligned} M_w &= 0.67 M_S + 2.07 && \text{for } 3.0 \leq M_S \leq 6.1 \\ M_w &= 0.99 M_S + 0.08 && \text{for } 6.2 \leq M_S \leq 8.2 \\ M_w &= 0.85 m_b + 1.03 && \text{for } 3.5 \leq m_b \leq 6.2 \end{aligned}$$

For M_L upto 5.7, the value of M_L was taken equal to M_w as suggested by Idriss (1985) and supported by operators of local networks in Pakistan. Conversion of M_L to M_w beyond magnitude 5.7 was done by using the following equations suggested by Ambraseys and Bommer (1990) and Ambraseys and Bilham (2003):

$$\begin{aligned} 0.82 (M_L) - 0.58 (M_S) &= 1.20 \\ \log M_o &= 19.09 + M_S && \text{for } M_S < 6.2 \\ \log M_o &= 15.94 + 1.5 M_S && \text{for } M_S > 6.2 \\ M_w &= (2/3) \log (M_o) - 10.73 \end{aligned}$$

Where m_b is body-wave magnitude, M_S is surface-wave magnitude, M_L is local magnitude, M_w is moment magnitude and M_o is seismic moment.

The composite earthquake list contains limited number of earthquakes prior to 1960 and only few of these earthquakes have been assigned magnitude values. Due to installation of WWSSN, the earthquake recording in this region improved and a better and complete recording of earthquake data are available after 1961. The completeness analysis of the overall data for Pakistan showed that earthquake data up to magnitude 4.5 is complete after 1960. The converted moment magnitude for the period between 1961 and 2006 was therefore used in the PSHA after excluding the aftershocks. A separate list of earthquakes occurring in each area source zone was prepared through GIS software and magnitude-frequency plots were made for each area source. The b -value for each area seismic source zone was calculated using linear regression through least square method. The minimum magnitude for each area source zone was selected from the magnitude-frequency curve based on completeness checks suggested by Woeffner and Weimer (2005).

A.3.4 Maximum Magnitude

To each area source zone, a maximum magnitude potential was assigned based on the maximum observed seismicity in the historical seismic record or enhancing by 0.5 magnitude the maximum observed magnitude in the instrumental seismic record for that area seismic source zone. For the fault characteristic model, the maximum magnitude of the fault was calculated using well known fault rupture-magnitude relationship developed by Well and Coppersmith (1994), taking half-length rupture. The maximum potential magnitude for Himalayan Frontal Thrust was selected equal to the magnitude of recent Kashmir earthquake which is considered the characteristic (maximum) for that fault.

A.3.5 Attenuation Equations

Because of lack of sufficient strong-motion data covering a larger range of magnitudes and distances, attenuation relationships for the South Asian Region could not be developed so far. A number of attenuation equations have been developed from strong motion data collected in other

parts of the world. As shallow earthquakes are of more concern for hazard analysis, attenuation equations developed for such conditions were considered for use in the hazard analysis. For probabilistic hazard analysis, the attenuation equation of Boore et al. (1997) was used as it incorporate the fault type and site characteristics in terms of shear wave velocity. Using this equation, the ground motions were calculated for rock site condition having shear wave velocity (v_s) of 760 m/sec. Three other equations developed during 1996 to 2004 along with Boore et al (1997) equation were also used in PSHA by giving equal weightage (25%) to each equation. Ground motion amplitudes obtained by these equations are generally same as obtained by using Boore et al. (1997) equation only.

A.3.6 Results of PSHA

The probabilistic hazard analysis was carried out by using EZ-FRISK software developed by Risk Engineering Inc. of Colorado, USA. As the purpose of the PSHA was to develop seismic hazard contour map, Gridded- Multisite module of EZ-FRISK software was used. In this module a probabilistic hazard analysis is performed for each point on a rectangular grid within the boundary of the region to be mapped. For ease of analysis, the hazard calculations were performed by dividing the study area (covering Pakistan and one degree outside Pakistan) into six parts and ground motion was obtained at each 0.1 degree interval of the rectangular grid (total about 13,000 grid points). The required parameters for all the seventeen area seismic source zones and twenty eight fault seismic sources (characteristic model) were fed to the software. The results of the hazard analysis obtained at each grid point are presented in the form of total hazard from all the seismic sources modeled (areas as well as faults) around 300 km radius of the grid point. The ground motion associated with 10% probability of exceedance in 50 years (475 years return period) was calculated at each grid point. From the results obtained at 0.1 degree interval, contours of Peak Ground Acceleration (PGA) values were plotted through GIS software to present the results in the form of seismic hazard map for 10% probability of exceedance in 50 years (i.e. 475 years return period). This PGA contour map is shown in Fig. A-1.

On the basis of PGA values obtained through PSHA, Pakistan was divided into five seismic zones in line with UBC (1997). The boundaries of these zones are defined on the following basis:

Zone 1	0.05 to 0.08g
Zone 2A	0.08 to 0.16g
Zone 2B	0.16 to 0.24g
Zone 3	0.24 to 0.32g
Zone 4	> 0.32g

The seismic zoning map of Pakistan developed on this basis is shown in Fig. 2.1. Each site shall be assigned a seismic zone in accordance with Fig. 2.1. Each structure shall be assigned a seismic zone factor Z in accordance with Table 5.9 given in Chapter 5.

A fault map of Pakistan prepared from the available geological maps and used for the present study is shown in Fig. A-2. The indicative locations of faults in areas designated as Zone 4 in Seismic Zoning Map of Pakistan are shown in Figs. A-3a to A-3e for general reference.

FIG. A-1

**Peak Ground Acceleration (g)
with 10% Probability of
Exceedance in 50 years**

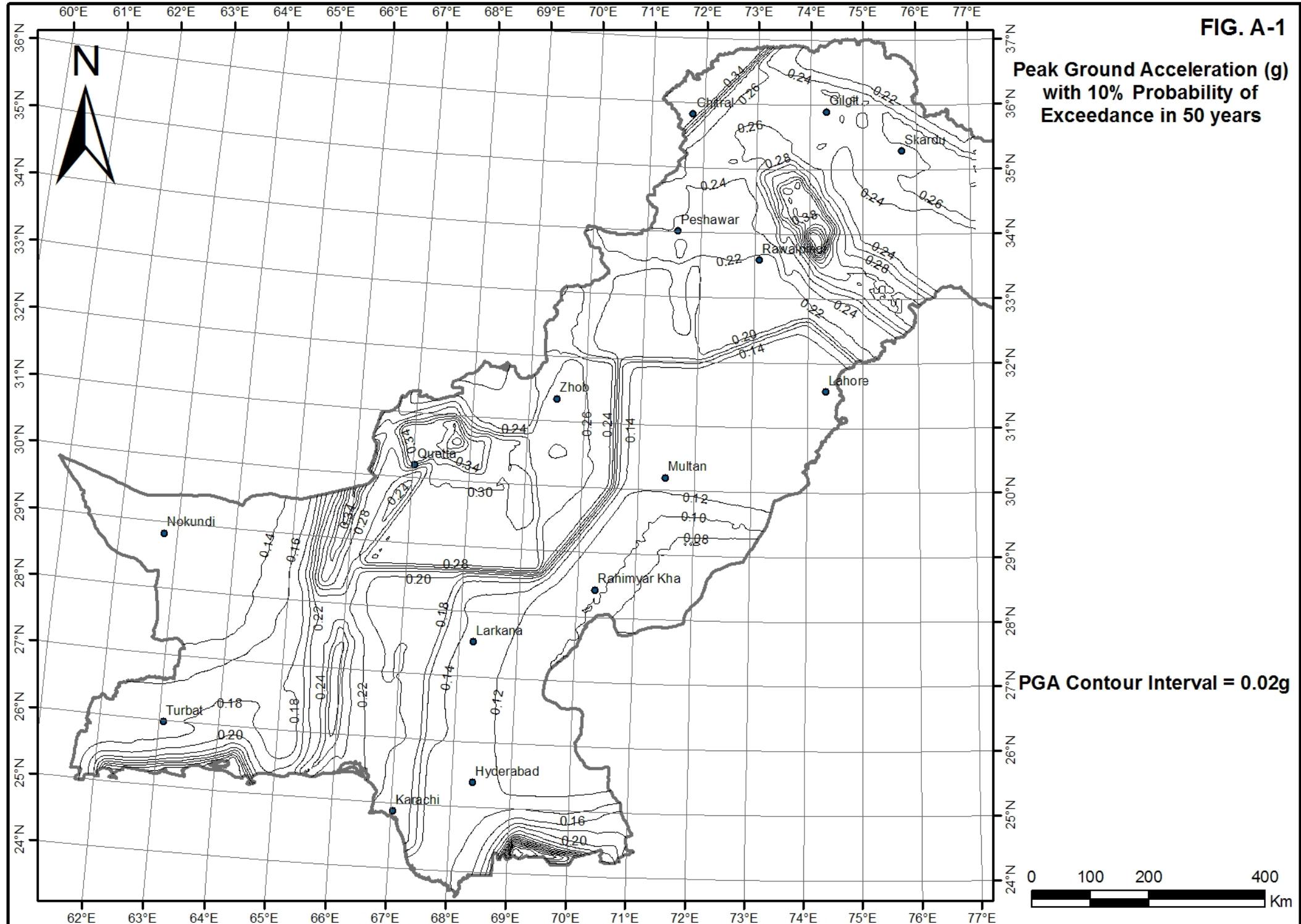


FIG. A-2

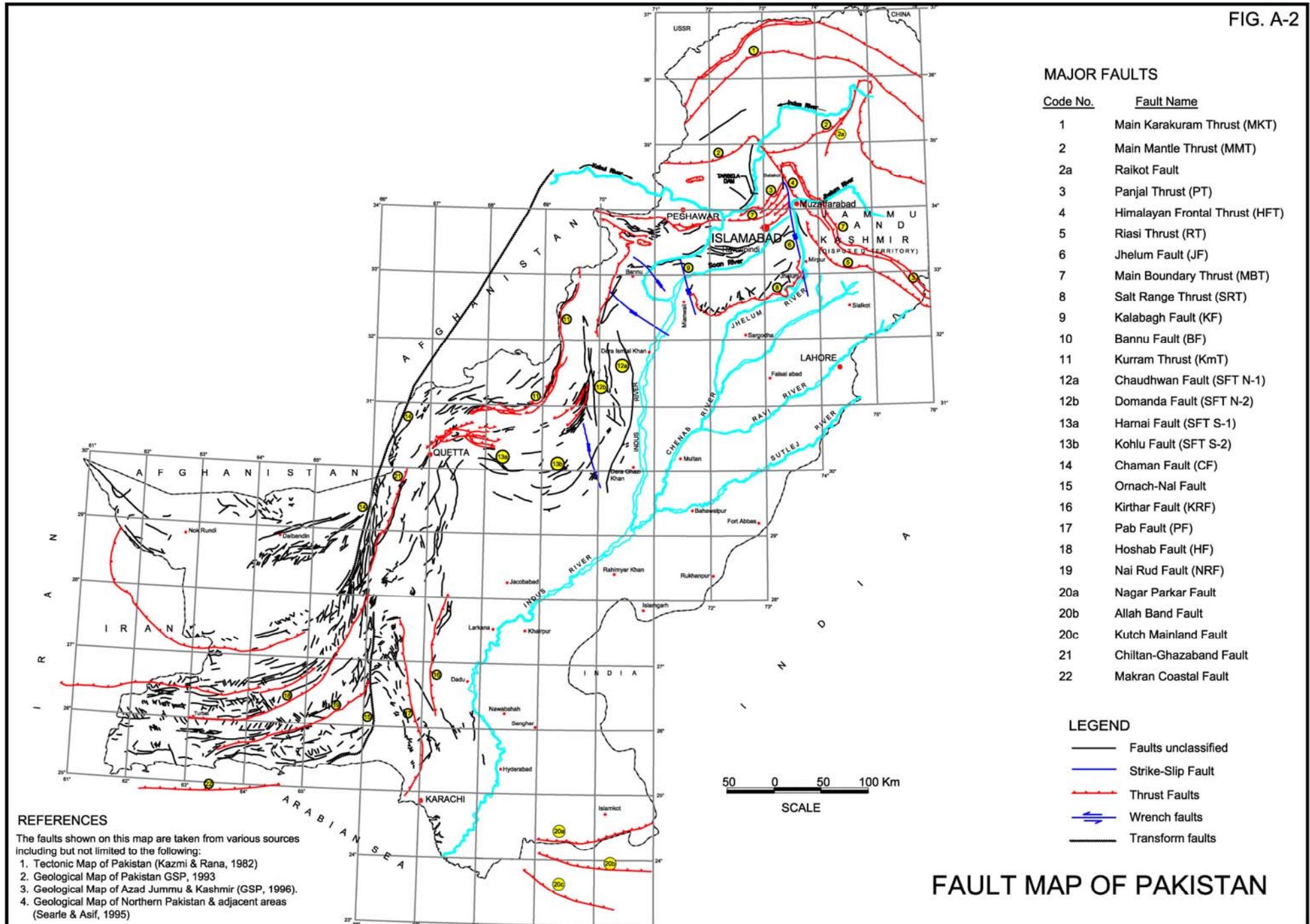
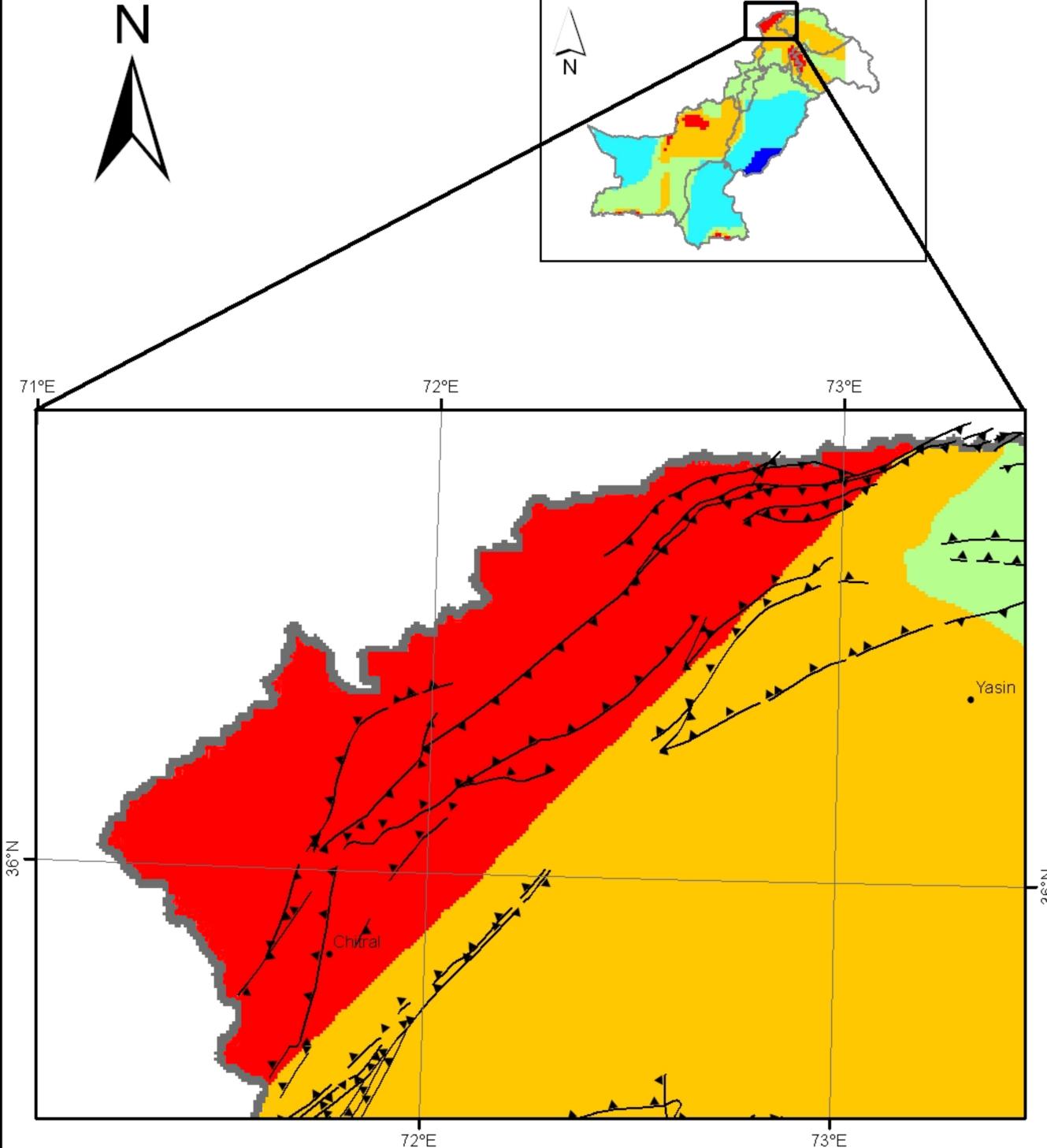


FIG. A-3a

DETAILS OF FAULTS IN ZONE 4



Note:

The location of faults are indicative only.
For exact location of faults with respect
to a site, refer to a qualified geologist or a
detailed geological map of the area.

0 15 30 60 Km

FIG. A-3b

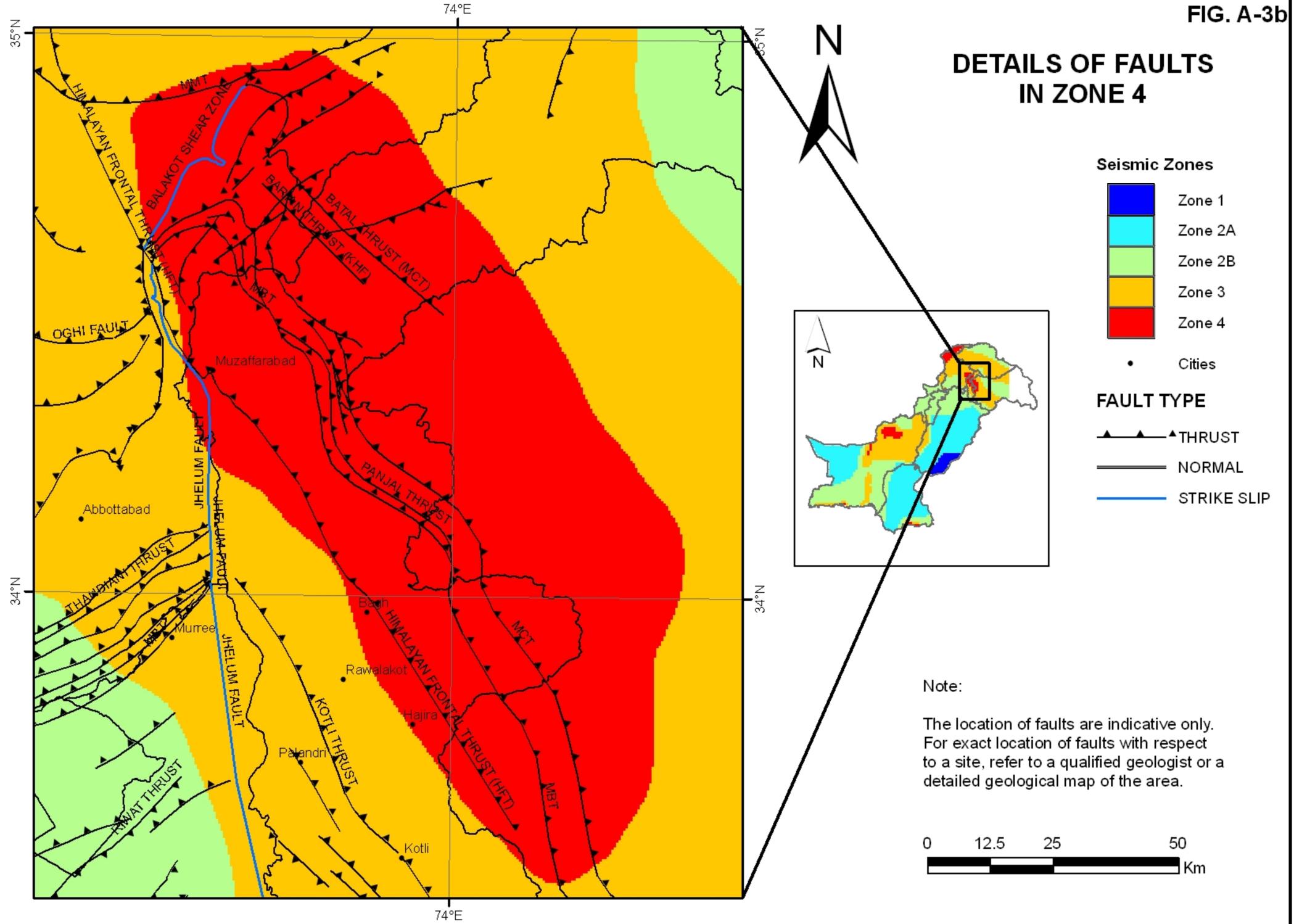


FIG. A-3c

DETAILS OF FAULTS IN ZONE 4

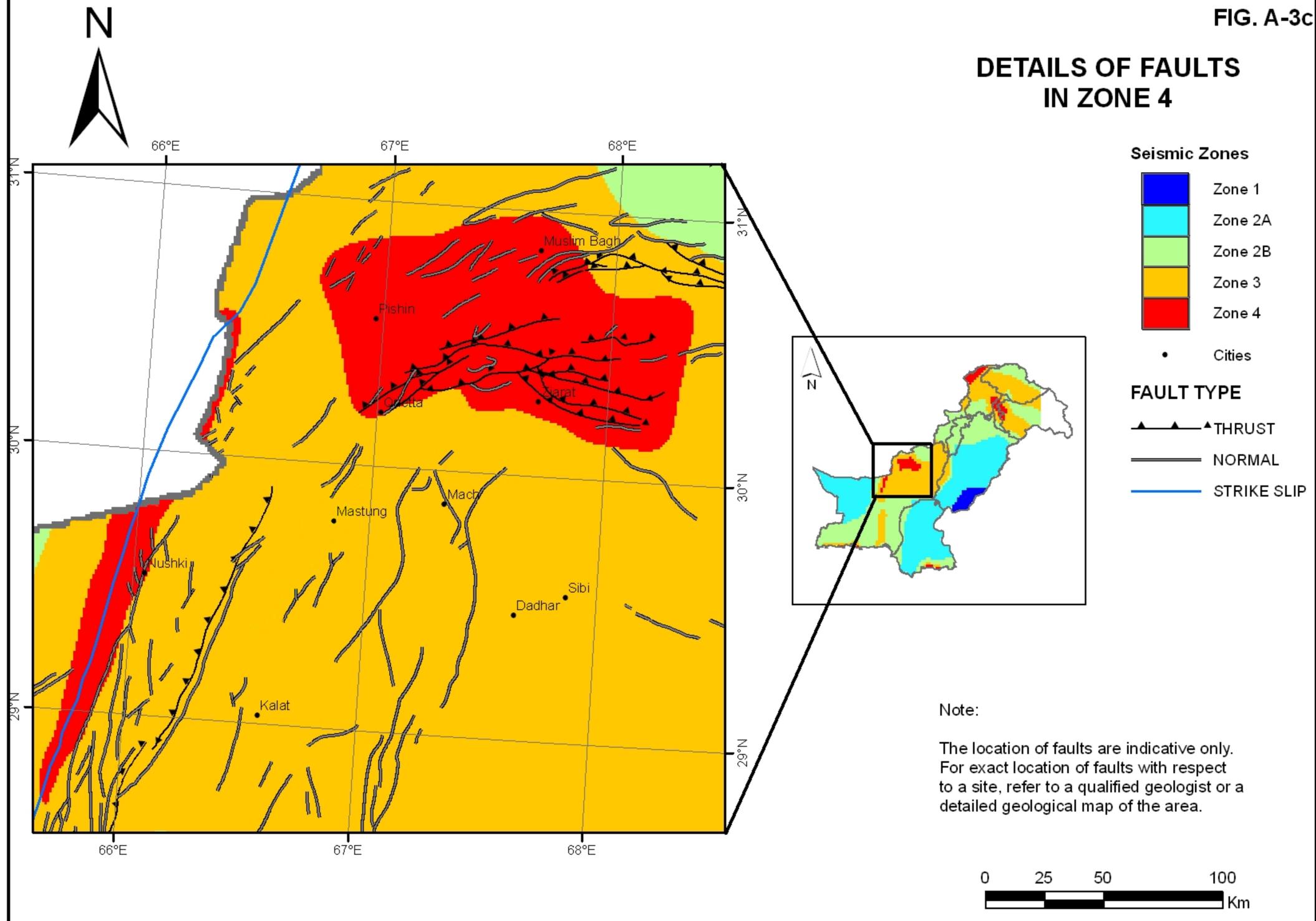


FIG. A-3d

DETAILS OF FAULTS IN ZONE 4

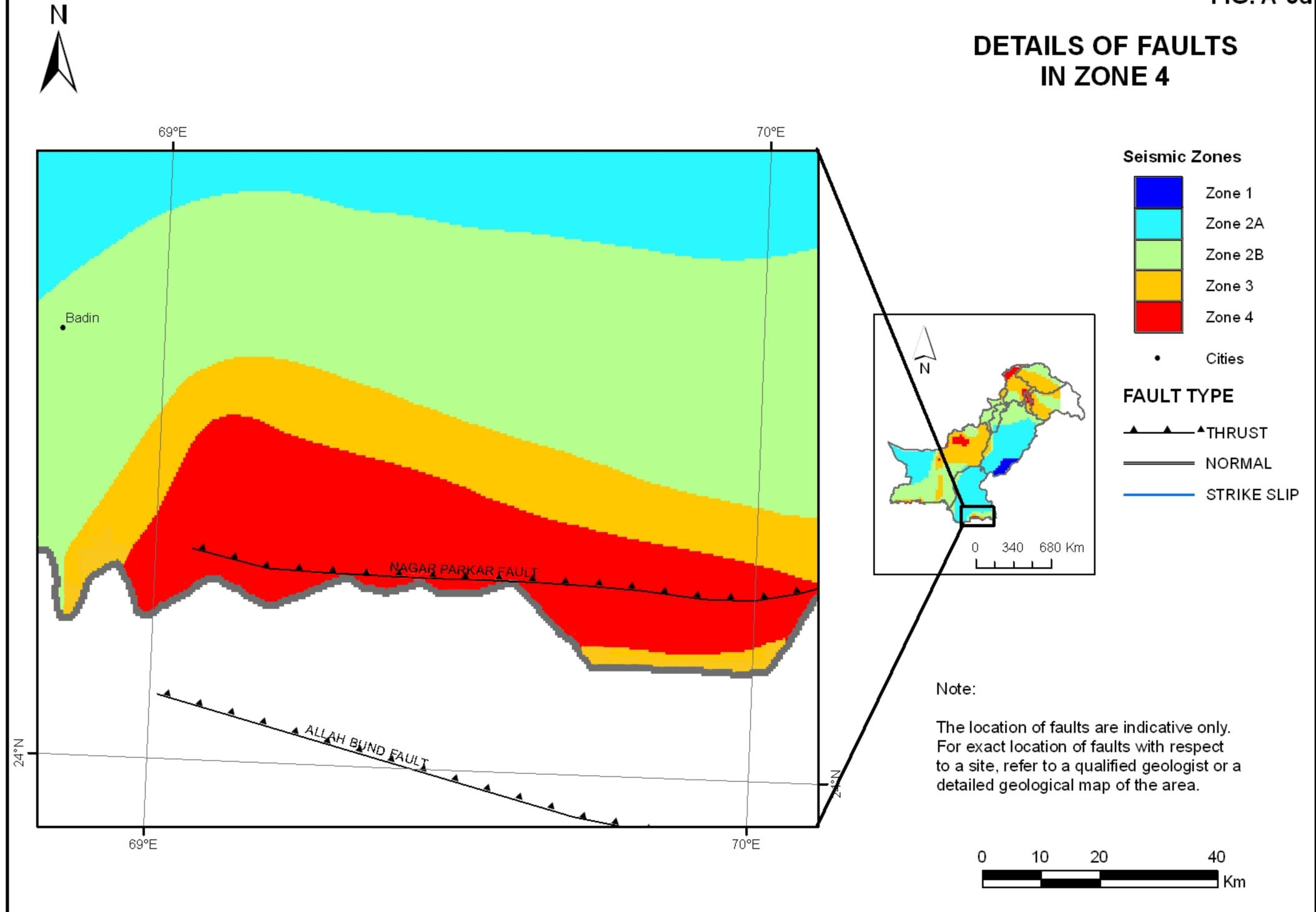
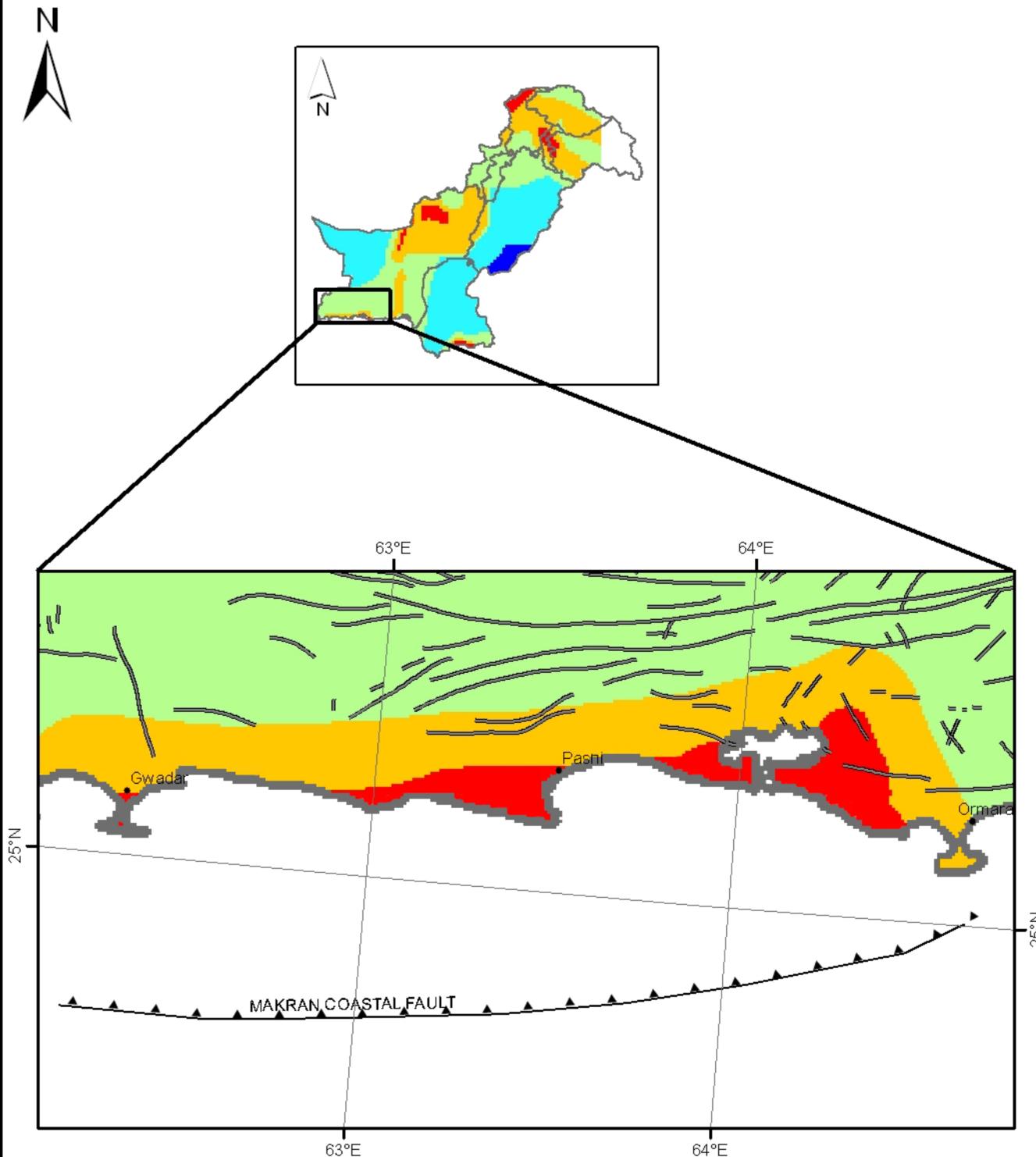


FIG. A-3e

DETAILS OF FAULTS IN ZONE 4



Note:

The location of faults are indicative only.
For exact location of faults with respect
to a site, refer to a qualified geologist or a
detailed geological map of the area.

0 20 40 80
Km