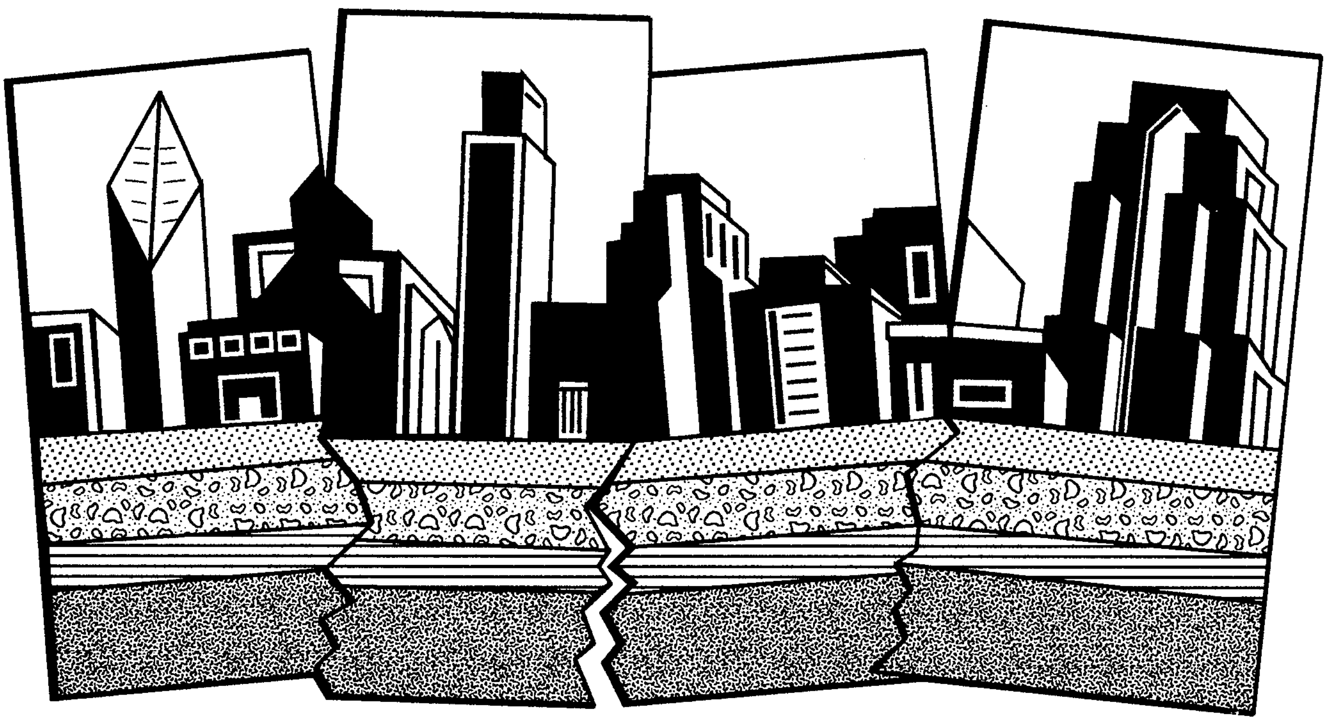
FEDERAL EMERGENCY MANAGEMENT AGENCY FEMA 369 / MARCH 2001

PB2002-100054

2000 Edition

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS AND OTHER STRUCTURES



Part 2 - Commentary

BUILDING SEISMIC SAFETY COUNCIL

Program on Improved Seismic Safety Provisions

Of the National Institute of Building Sciences

2000 Edition

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS AND OTHER STRUCTURES

Part 2: Commentary (FEMA 369)

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

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings.

To fulfill its purpose, the BSSC: (1) promotes the development of seismic safety provisions suitable for use throughout the United States; (2) recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes; (3) assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies; (4) identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements; (5) promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, elected officials, industry representatives, other members of the building community, and the public; (6) advises government bodies on their programs of research, development, and implementation; and (7) periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

See the back of this *Commentary* volume for a full description of BSSC activities.

BSSC Program on Improved Seismic Safety Provisions

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

2000 EDITION

Part 2: COMMENTARY
(FEMA 369)

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

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
2001

This report was prepared under Contract EMW-97-CO-0481 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

Building Seismic Safety Council activities and products are described at the end of this report. For further information, contact the Building Seismic Safety Council, 1090 Vermont, Avenue, N.W., Suite 700, Washington, D.C. 20005; phone 202-289-7800; fax 202-289-1092; e-mail bssc@nibs.org.

Copies of this report may be obtained by contacting the FEMA Publication Distribution Facility at 1-800-480-2520.

The National Institute of Building Sciences and its Building Seismic Safety Council caution users of these *Provisions* documents to be alert to patent and copyright concerns especially when applying prescriptive requirements.

CONTENTS

Chapter 1 Commentary, GENERAL PROVISIONS 1

1.1 PURPOSE 2

1.2.1 Scope 3

1.2.2 Additions 4

1.2.3 Change of Use 4

1.2.4 Alterations 4

1.2.5 Alternate Materials and Alternate Means and Methods of Construction 4

1.3 SEISMIC USE GROUPS 5

1.3.5 Seismic Use Group III Structure Access Protection 9

1.4 OCCUPANCY IMPORTANCE FACTOR 9

Chapter 2 Commentary, GLOSSARY AND NOTATIONS 11

2.1 GLOSSARY 11

2.2 NOTATIONS 20

Chapter 3 Commentary, QUALITY ASSURANCE 39

3.1 SCOPE 39

3.2 QUALITY ASSURANCE 40

3.3 SPECIAL INSPECTION 41

3.3.9 Architectural Components 42

3.3.10 Mechanical and Electrical Components 42

3.4 TESTING 42

3.4.5 Mechanical and Electrical Equipment 42

3.5 STRUCTURAL OBSERVATIONS 43

3.6 REPORTING AND COMPLIANCE PROCEDURES 43

Chapter 4 Commentary, GROUND MOTION 45

4.1 PROCEDURES FOR DETERMINING MAXIMUM CONSIDERED EARTHQUAKE AND DESIGN EARTHQUAKE GROUND MOTION ACCELERATIONS AND RESPONSE SPECTRA 45

4.1.1 Maximum Considered Earthquake Ground Motions 45

4.1.2 General Procedure for Determining Maximum Considered Earthquake Ground

Motions and Design Spectral Response Accelerations 46
4.2 SEISMIC DESIGN CATEGORY 62
4.2.2 Site Limitation for Seismic Design Categories E and F 64

Chapter 5 Commentary, STRUCTURAL DESIGN CRITERIA 69
5.1 REFERENCE DOCUMENT 69
5.2 DESIGN BASIS 69
5.2.2 Basic Seismic-Force-Resisting Systems 73
5.2.3 Structure Configuration 79
5.2.4 Redundancy 83
5.2.5 Structural Analysis 84
5.2.6 Design and Detailing Requirements 88
5.2.7 Combination of Load Effects 93
5.2.8 Deflection and Drift Limits 99
5.3 INDEX FORCE ANALYSIS PROCEDURE 101
5.4 EQUIVALENT LATERAL FORCE PROCEDURE 101
5.4.1 Seismic Base Shear 102
5.4.2 Period Determination 104
5.4.3 Vertical Distribution of Seismic Forces 108
5.4.4 Horizontal Shear Distribution 109
5.4.5 Overturning 112
5.4.6 Drift Determination and *P*-delta Effects 112
5.5 MODAL RESPONSE SPECTRUM ANALYSIS PROCEDURE 115
5.5.1 General 115
5.5.2 Modes 115
5.5.3 Modal Properties 116
5.5.4 Modal Base Shear 116
5.5.5 Modal Forces, Deflections, and Drifts 117
5.5.6 Modal Story Shears and Moments 117
5.5.7 Design Values 117
5.5.8 Horizontal Shear Distribution 118
5.5.9 Foundation Overturning 118

5.5.10 *P*-Delta Effects 118
5.6 LINEAR RESPONSE HISTORY ANALYSIS PROCEDURE 118
5.7 NONLINEAR RESPONSE HISTORY ANALYSIS 119
5.7.4 Design Review 121
5.8 SOIL-STRUCTURE INTERACTION EFFECTS 121
5.8.1 General 121
5.8.2 Equivalent Lateral Force Procedure 126
5.8.3 Modal Analysis Procedure 140
Appendix to Chapter 5
NONLINEAR STATIC ANALYSIS PROCEDURE 147

Chapter 6 Commentary, ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS 153
6.1 GENERAL 153
6.1.2 Component Force Transfer 154
6.1.3 Seismic Forces 155
6.1.4 Seismic Relative Displacements 159
6.1.5 Component Importance Factor 160
6.1.6 Component Anchorage 161
6.1.7 Construction Documents 162
6.2 ARCHITECTURAL COMPONENT DESIGN 163
6.2.1 General 163
6.2.2 Architectural Component Forces and Displacements 163
6.2.3 Architectural Component Deformation 164
6.2.4 Exterior Nonstructural Wall Elements and Connections 164
6.2.5 Out-of-Plane Bending 165
6.2.6 Suspended Ceilings 165
6.2.7 Access Floors 166
6.2.8 Partitions 167
6.2.9 Steel Storage Racks 167
6.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions . . 167
6.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN 177
6.3.1 General 177

6.3.2 Mechanical and Electrical Component Forces and Displacements 179
6.3.3 Mechanical and Electrical Component Period 179
6.3.4 Mechanical and Electrical Component Attachments 180
6.3.5 Component Supports 180
6.3.6 Component Certification 181
6.3.7 Utility and Service Lines at Structure Interfaces 181
6.3.9 Storage Tanks 181
6.3.10 HVAC Ductwork 182
6.3.11 Piping Systems 182
6.3.12 Boilers and Pressure Vessels 182
6.3.13 Mechanical Equipment Attachments and Supports 183
6.3.14 Electrical Equipment Attachments and Supports 183
6.3.15 Alternate Seismic Qualification Methods 183
6.3.16 Elevator Design Requirements 183

Chapter 7 Commentary, FOUNDATION DESIGN REQUIREMENTS 187
7.1 GENERAL 187
7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS 187
7.2.1 Structural Materials 187
7.2.2 Soil Capacities 187
7.3 SEISMIC DESIGN CATEGORIES A AND B 187
7.4 SEISMIC DESIGN CATEGORY C 187
7.4.1 Investigation 187
7.4.2 Pole-Type Structures 202
7.4.3 Foundation Ties 202
7.4.4 Special Pile Requirements 202
7.5 SEISMIC DESIGN CATEGORIES D , E, AND F 204
7.5.1 Investigation 205
7.5.2 Foundation Ties 209
7.5.4 Special Pile and Grade Beam Requirements 209

Chapter 8 Commentary, STEEL STRUCTURE DESIGN REQUIREMENTS 215

8.1 REFERENCE DOCUMENTS 215
8.2 SEISMIC REQUIREMENTS FOR STEEL STRUCTURES 215
8.3 SEISMIC DESIGN CATEGORIES A, B, AND C 215
8.4 SEISMIC DESIGN CATEGORIES D, E, AND F 215
8.5 COLD-FORMED STEEL SEISMIC REQUIREMENTS 215
8.6 LIGHT-FRAMED WALLS 215
8.7 SEISMIC REQUIREMENTS FOR STEEL DECK DIAPHRAGMS 216
8.8 STEEL CABLES 216

Chapter 9 Commentary, CONCRETE STRUCTURE DESIGN REQUIREMENTS 217
9.1 REFERENCE DOCUMENT 217
9.1.1 Modifications to ACI 318 217
9.2 ANCHORING TO CONCRETE 222
9.2.1 Scope 222
9.2.2 Notations and Definitions 223
9.2.3 General Requirements 224
9.2.4 General Requirements for Strength of Structural Anchors 225
9.2.5 Design Requirements for Tensile Loading 230
9.2.6 Design Requirements for Shear Loading 235
9.2.7 Interaction of Tensile and Shear Forces 241
9.2.8 Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure 241
9.2.9 Installation of Anchors 242
9.3 CLASSIFICATION OF SHEAR WALLS 242
9.4 SEISMIC PERFORMANCE CATEGORY A 242
9.5 SEISMIC PERFORMANCE CATEGORY B 242
9.5.1 Ordinary Moment Frames 243
9.6 SEISMIC PERFORMANCE CATEGORY C 243
9.6.2 Intermediate Moment Frames and 9.6.3 Special Moment Frames 243
9.7 SEISMIC PERFORMANCE CATEGORIES D, E, or F 244
Appendix to Chapter 9
REINFORCED CONCRETE DIAPHRAGMS CONSTRUCTED USING UNTOPPED PRECAST CONCRETE ELEMENTS 250

Chapter 10 Commentary, COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS 253

Chapter 11 Commentary, MASONRY STRUCTURE DESIGN REQUIREMENTS 255

11.1 GENERAL 255

11.1.1 Scope 255

11.1.2 Reference Documents 255

11.1.3 Definitions 255

11.1.4 Notations 255

11.2 CONSTRUCTION REQUIREMENTS 255

11.2.1 General 255

11.2.2 Quality Assurance 255

11.3 GENERAL REQUIREMENTS 256

11.3.1 Scope 256

11.3.2 Empirical Masonry Design 256

11.3.3 Plain (Unreinforced) Masonry Design 256

11.3.4 Reinforced Masonry Design 256

11.3.5 - 11.3.9 Seismic Design Categories A through F 257

11.3.6 Seismic Design Category B 257

11.3.10: Properties of Materials 257

11.3.11 Section Properties 257

11.3.12 Headed and Bent-Bar Anchor Bolts 258

11.4 DETAILS OF REINFORCEMENT 258

11.4.5 Development of Reinforcement 258

11.5 STRENGTH AND DEFORMATION REQUIREMENTS 259

11.5.3 Design Strength 259

11.5.4 Deformation Requirements 260

11.6 FLEXURE AND AXIAL LOADS 260

11.6.2 Design Requirements of Reinforced Masonry Members 260

11.6.3 Design of Plan (Unreinforced) Masonry Members 264

11.7 SHEAR 265

11.7.3 Design of Reinforced Masonry Members 265
11.8 SPECIAL REQUIREMENTS FOR BEAMS 267
11.8.4 Deep Flexural Members 268
11.9 SPECIAL REQUIREMENTS FOR COLUMNS 269
11.10 SPECIAL REQUIREMENTS FOR SHEAR WALLS 269
11.10.6 Flanged Shear Walls 269
11.10.7 Coupled Shear Walls 270
11.12 GLASS-UNIT MASONRY AND MASONRY VENEER 270

Chapter 12 Commentary, WOOD STRUCTURE DESIGN REQUIREMENTS 277
12.1.2 REFERENCE DOCUMENTS 277
12.1.3 Notations 277
12.2 DESIGN METHODS 278
12.3 GENERAL DESIGN REQUIREMENTS FOR ENGINEERED WOOD CONSTRUCTION 279
12.3.2 Shear Resistance Based on Principles of Mechanics 280
12.3.3 Deformation Compatibility Requirements 280
12.3.4 Framing Requirements 280
12.3.5 Sheathing Requirements 281
12.3.6 Wood Members Resisting Horizontal Seismic Forces Contributed by Masonry and Concrete 282
12.4 DIAPHRAGMS AND SHEAR WALLS 282
12.4.1 Diaphragms 285
12.4.2 Shear Walls 288
12.4.3 Perforated Shear Walls 293
12.5 CONVENTIONAL LIGHT-FRAME CONSTRUCTION 306
12.5.2 Braced Walls 308
12.5.3 Detailing Requirements 309
12.6 SEISMIC DESIGN CATEGORY A 325
12.7 SEISMIC DESIGN CATEGORIES B, C, AND D 325
12.8 SEISMIC DESIGN CATEGORIES E and F 325

Chapter 13 Commentary, SEISMICALLY ISOLATED STRUCTURES DESIGN

REQUIREMENTS . . . 329
13.1 GENERAL . . . 331
13.2 CRITERIA SELECTION . . . 332
13.3 EQUIVALENT LATERAL FORCE PROCEDURE . . . 334
13.3.3 Minimum Lateral Displacements . . . 334
13.3.4 Minimum-Lateral Forces . . . 335
13.3.5 Vertical Distribution of Force . . . 336
13.3.6 Drift Limits . . . 336
13.4 DYNAMIC LATERAL RESPONSE PROCEDURE . . . 337
13.5 LATERAL LOAD ON ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS SUPPORTED BY BUILDINGS . . . 338
13.6 DETAILED SYSTEM REQUIREMENTS . . . 338
13.6.3 Structural System . . . 339
13.8 DESIGN AND CONSTRUCTION REVIEW . . . 339
13.9 REQUIRED TESTS OF THE ISOLATION SYSTEM . . . 339
13.9.5 Design Properties of the Isolated System . . . 342
Appendix to Chapter 13 Commentary
STRUCTURES WITH DAMPING SYSTEMS . . . 344

Chapter 14 Commentary, NONBUILDING STRUCTURE DESIGN REQUIREMENTS . . . 351
14.1 GENERAL . . . 351
14.1.1 Scope . . . 351
14.2 REFERENCES . . . 352
14.4 NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES . . . 353
14.5 STRUCTURAL DESIGN REQUIREMENTS . . . 353
14.5.1 Design Basis . . . 353
14.5.2 Rigid Nonbuilding Structures . . . 357
14.5.4 Fundamental Period . . . 357
14.6 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS . . . 357
14.6.2 Pipe Racks . . . 358
14.6.3 Steel Storage Racks . . . 358
14.6.4 Electrical Power Generating Facilities . . . 358
14.6.6 Piers and Wharves . . . 358

14.7 NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS 358
14.7.1 General 359
14.7.2 Earth Retaining Structures 359
14.7.3 Tanks and Vessels 359
14.7.4 Stacks and Chimneys 367
Appendix to Chapter 14 368

Commentary Appendix A, DEVELOPMENT OF MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAPS 1 THROUGH 24 375

Commentary Appendix B, DEVELOPMENT OF THE USGS SEISMIC MAPS 391

THE COUNCIL: ITS PURPOSE AND ACTIVITIES 431

BSSC MEMBER ORGANIZATIONS 443

BUILDING SEISMIC SAFETY COUNCIL PUBLICATIONS 444

Chapter 1 Commentary

GENERAL PROVISIONS

Chapter 1 sets forth general requirements for applying the analysis and design provisions contained in Chapters 2 through 14 of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. It is similar to what might be incorporated in a code as administrative regulations.

Chapter 1 is designed to be as compatible as possible with normal code administrative provisions (especially as exemplified by the three national model codes), but it is written as the guide to use of the rest of the document, not as a regulatory mechanism. The word "shall" is used in the *Provisions* not as a legal imperative, but simply as the language necessary to ensure fulfillment of all the steps necessary to technically meet a minimum standard of performance.

It is important to note that the *Provisions* is intended to serve as a resource document for use by any interested member of the building community. Thus, some users may alter certain information within the *Provisions* (e.g., the determination of which use groups are included within the higher Seismic Use Groups might depend on whether the user concluded that the generally more-demanding design requirements were necessary). It is strongly emphasized, however, that such "tailoring" should be carefully considered by highly qualified individuals who are fully aware of all the implications of any changes on all affected procedures in the analysis and design sequences of the document.

Further, although the *Provisions* is national in scope, it presents minimum criteria. It is neither intended to nor does it justify any reduction in higher standards that have been locally established, particularly in areas of highest seismicity.

Reference is made throughout the document to decisions and actions that are delegated to an unspecified "authority having jurisdiction." The document is intended to be applicable to many different types of jurisdictions and chains of authority, and an attempt has been made to recognize situations where more than technical decision-making can be presumed. In fact, the document anticipates the need to establish standards and approval systems to accommodate the use of the document for development of a regulatory system. A good example of this is in Sec. 1.2.6, Alternate Materials and Alternate Means and Methods of Construction, where the need for well-established criteria and systems of testing and approval are recognized even though few such systems are in place. In some instances, the decision-making mechanism referred to is clearly most logically the province of a building official or department; in others, it may be a law-making body such as a state legislature, a city council, or some other state or local policy-making body. The term "authority having jurisdiction" has been used to apply to all of these entities. A good example of the need for keeping such generality in mind is provided by the California law concerning the design and construction of schools. That law establishes requirements for independent special inspection approved and supervised by the Office of the State Architect, a state-level office that does not exist in many other states.

Note that Appendix A to this *Commentary* volume presents a detailed explanation of the development of *Provisions* Maps 1 through 24 and Appendix B describes development of the U.S. Geological Survey seismic hazard maps on which the *Provisions* maps are based. An overview of the Building Seismic Safety Council (BSSC) and its activities appears at the end of the volume.

1.1 PURPOSE: The goal of the *Provisions* is to present criteria for the design and construction of new structures subject to earthquake ground motions in order to minimize the hazard to life for all structures, to increase the expected performance of structures having a substantial public hazard due to occupancy or use as compared to ordinary structures, and to improve the capability of essential facilities to function after an earthquake. To this end, the *Provisions* provides the minimum criteria considered prudent for the protection of life safety in structures subject to earthquakes. The *Provisions* document has been reviewed extensively and balloted by the architectural, engineering, and construction communities and, therefore, it is a proper source for the development of building codes in areas of seismic exposure.

Some design standards go farther than the *Provisions* and attempt to minimize damage as well as protect building occupants. For example, the *California Building Code* has added property protection in relation to the design and construction of hospitals and public schools. The *Provisions* document generally considers property damage as it relates to occupant safety for ordinary structures. For high occupancy and essential facilities, damage limitation criteria are more strict in order to better provide for the safety of occupants and the continued functioning of the facility.

Some structural and nonstructural damage can be expected as a result of the "design ground motions" because the *Provisions* allow inelastic energy dissipation in the structural system. For ground motions in excess of the design levels, the intent of the *Provisions* is for the structure to have a low likelihood of collapse.

It must be emphasized that absolute safety and no damage even in an earthquake event with a reasonable probability of occurrence cannot be achieved for most structures. However, a high degree of life safety, albeit with some structural and nonstructural damage, can be economically achieved in structures by allowing inelastic energy dissipation in the structure. The objective of the *Provisions* therefore is to set forth the minimum requirements to provide reasonable and prudent life safety. For most structures designed and constructed according to the *Provisions*, it is expected that structural damage from even a major earthquake would likely be repairable, but the damage may not be economically repairable.

Where damage control is desired, the design must provide not only sufficient strength to resist the specified seismic loads but also the proper stiffness to limit the lateral deflection. Damage to nonstructural elements may be minimized by proper limitation of deformations; by careful attention to detail; and by providing proper clearances for exterior cladding, glazing, partitions, and wall panels. The nonstructural elements can be separated or floated free and allowed to move independently of the structure. If these elements are tied rigidly to the structure, they should be protected from deformations that can cause cracking; otherwise, one must expect such damage. It should be recognized, however, that major earthquake ground motions can cause deformations much larger than the specified drift limits in the *Provisions*.

Where prescribed wind loading governs the stress or drift design, the resisting system still must conform to the special requirements for seismic force resisting systems. This is required in order to resist, in a ductile manner, potential seismic loadings in excess of the prescribed loads.

A proper continuous load path is an obvious design requirement for equilibrium, but experience has shown that it often is overlooked and that significant damage and collapse can result. The basis for this design requirement is twofold:

1. To ensure that the design has fully identified the seismic force resisting system and its appropriate design level and
2. To ensure that the design basis is fully identified for the purpose of future modifications or changes in the structure.

Detailed requirements for selecting or identifying and designing this load path are given in the appropriate design and materials chapters.

1.2.1 Scope: The scope statement establishes in general terms the applicability of the *Provisions* as a base of reference. Certain *structures* are exempt and need not comply:

1. Detached one- and two-family dwellings in *Seismic Design Categories* A, B, and C are exempt because they represent low seismic risks.
2. *Structures* constructed using the conventional light-frame construction requirements in Sec. 12.5 are deemed capable of resisting the *seismic forces* imposed by the *Provisions.* While specific elements of conventional light-frame construction may be calculated to be over-stressed, there is typically a great deal of redundancy and uncounted resistance in such *structures*. Detached one- and two-story wood frame dwellings have generally performed well even in regions of higher seismicity. The requirements of Sec. 12.5 are adequate to provide the safety required for such dwellings without imposing any additional requirements of the *Provisions*.
3. Agricultural storage *structures* are generally exempt from most code requirements because of the exceptionally low risk to life involved and that is the case of the *Provisions*.
4. *Structures* in areas with extremely low seismic risk need only comply with the design and detailing requirements for *structures* assigned to *Seismic Design Category* A.

The *Provisions* are not retroactive and apply only to existing *structures* when there is an *addition*, change of use, or *alteration*. As a minimum, existing *structures* should comply with legally adopted regulations for repair and rehabilitation as related to earthquake resistance. (Note: Publications such as the Seismic Rehabilitation Guidelines and Commentary- FEMA 273 &274 are available.)

The *Provisions* are not written to prevent damage due to earth slides (such as those that occurred in Anchorage, Alaska), to liquefaction (such as occurred in Niigata, Japan), or to tsunami (such as occurred in Hilo, Hawaii). It provides for only minimum required resistance to earthquake ground-shaking, without settlement, slides, subsidence, or faulting in the immediate vicinity of the *structure*.

1.2.2 Additions: Additions that are structurally independent of an existing structure are considered to be new structures required to conform with the *Provisions*. For additions that are not structurally independent, the intent is that the addition as well as the existing structure be made to comply with the *Provisions* except that an increase of up to 5 percent of the mass contributing to seismic forces is permitted in any elements of the existing structure without bringing the entire structure into conformance with the *Provisions*. Additions also shall not reduce the lateral force resistance of any existing element to less than that required for a new structure.

1.2.3 Change of Use: When a change in the use of a *structure* will result in the *structure* being reclassified to a higher *Seismic Use Group*, the existing *structure* must be brought into compliance with the requirements of the *Provisions* as if it were a new *structure*. *Structures* in higher *Seismic Use Groups* are intended to provide a higher level of safety to occupants and in the case of *Seismic Use Group* III be capable of performing their safety-related function after a seismic event. An exception is allowed when the change is from *Seismic Use Group* I to *Seismic Use Group* II where S_{DS} is less than 0.3. The expense that may be necessary to upgrade such as structure because of a change in the Seismic Use Group cannot be justified for structures located in regions with low seismic risk.

1.2.4 Alterations: *Alterations* include all significant modifications to existing *structures* that are not classified as an *addition*. No reduction in strength of the *seismic-force-resisting system* or stiffness of the *structure* shall result from an *alteration* unless the altered *structure* is determined to be in compliance with the *Provisions*. Like *additions*, an increase of not greater than 5 percent of the mass contributing to *seismic forces* is permitted in any structural element of the existing *structure* without bringing the entire *structure* into conformance with the *Provisions*.

The cumulative effects of *alterations* and *additions* should not increase the *seismic forces* in any structural element of the existing *structure* by more than 5 percent unless the capacity of the element subject to the increased *seismic forces* is still in compliance with the *Provisions*.

1.2.5 Alternate Materials and Alternate Means and Methods of Construction: It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction either existing or anticipated. While not citing specific materials or methods of construction currently available that require approval, this section serves to emphasize the fact that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgment of the best use of the materials and methods based on well-established expertise and historical seismic performance. It is important that any replacement or substitute be evaluated with an understanding of all the ramifications of performance, strength, and durability implied by the *Provisions*.

It also is recognized that until needed approval standards and agencies are created, authorities having jurisdiction will have to operate on the basis of the best evidence available to substantiate any application for alternates. If accepted standards are lacking, it is strongly recommended that applications be supported by extensive reliable data obtained from tests simulating, as closely as is practically feasible, the actual load and/or deformation conditions to which the material is expected to be subjected during the service life of the structure. These conditions, where applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

1.3 SEISMIC USE GROUPS: The expected performance of *structures* shall be controlled by assignment of each *structure* to one of three *Seismic Use Groups. Seismic Use Groups* are categorized based on the occupancy of the *structures* within the group and the relative consequences of earthquake induced damage to the *structures.* The *Provisions* specify progressively more conservative strength, drift control, system selection and detailing requirements for *structures* contained in the three groups, in order to attain minimum levels of earthquake performance suitable to the individual occupancies.

In previous editions of the *Provisions*, this categorization of *structures*, by occupancy, or use, was termed a Seismic Hazard Exposure Group. The name *Seismic Use Group* was adopted in the 1997 *Provisions* as being more representative of the definition of this classification. Seismic hazard relates to the severity and frequency of ground motion expected to affect a *structure*. Since *structures* contained in these groups are spread across the various zones of seismicity, from high to low hazard, the groups do not really relate to hazard. Rather the groups, categorized by occupancy or use, are used to establish design criteria intended to produce specific types of performance in design earthquake events, based on the importance of reducing structural damage and improving life safety.

In terms of post-earthquake recovery and redevelopment, certain types of occupancies are vital to public needs. These special occupancies were identified and given specific recognition. In terms of disaster preparedness, regional communication centers identified as critical emergency services should be in a higher classification than retail stores, office buildings, and factories.

Specific consideration is given to Group III, essential facilities required for post-earthquake recovery. Also included are *structures* that contain substances, that if released into the environment, are deemed to be hazardous to the public. The 1991 Edition included a flag to urge consideration of the need for utility services after an earthquake. It is at the discretion of the authority having jurisdiction which *structures* are required for post-earthquake response and recovery. This is emphasized with the term "designated" before many of the *structures* listed in Sec. 1.3.1. Using Item 3, "designated medical facilities having emergency treatment facilities" as an example, the authority having jurisdiction should inventory medical facilities having emergency treatment facilities within the jurisdiction and designate those to be required for post-earthquake response and recovery. In a rural location where there may not be a major hospital, the authority having jurisdiction may choose to require outpatient surgery clinics to be designated Group III *structures*. On the other hand, these same clinics in a major jurisdiction with hospitals nearby may not need to be designated Group III *structures*.

Group II *structures* are those having a large number of occupants and those where the occupants ability to exit is restrained. The potential density of public assembly uses in terms of number of people warrant an extra level of care. The level of protection warranted for schools, day care centers, and medical facilities is greater than the level of protection warranted for occupancies where individuals are relatively self-sufficient in responding to an emergency.

Group I contains all uses other than those excepted generally from the requirements in Sec. 1.2. Those in Group I have lesser life hazard only insofar as there is the probability of lesser numbers of occupants in the *structures* and the *structures* are lower and/or smaller.

In *structures* with multiple uses, the 1988 Edition of the *Provisions* required that the *structure* be assigned the classification of the highest group occupying 15 percent or more of the total area of the *structure.* This was changed in the 1991 Edition to require the *structure* to be assigned to the highest group present. These requirements were further modified to allow different portions of a *structure* to be assigned different *Seismic Use Groups* provided the higher group is not negatively impacted by the lower group. When a lower group impacts a higher group, the higher group must either be seismically independent of the other, or the two must be in one *structure* designed seismically to the standards of the higher group. Care must be taken, however, for the case in which the two uses are seismically independent but are functionally dependent. The fire and life-safety requirements relating to exiting, occupancy, fire-resistive construction and the like of the higher group must not be reduced by interconnection to the lower group. Conversely, one must also be aware that there are instances, although uncommon, where certain fire and life-safety requirements for a lower group may be more restrictive than those for the higher group. Such assignments also must be considered when changes are made in the use of a *structure* even though existing *structures* are not within the scope of the *Provisions.*

Consideration has been given to reducing the number of groupings by combining Groups I and II and leaving Group III the same as is stated above; however, the consensus of those involved in the *Provisions* development and update efforts to date is that such a merging would not be responsive to the relative performance desired of *structures* in these individual groups.

Although the *Provisions* explicitly require design for only a single level of ground motion, it is expected that *structures* designed and constructed in accordance with these requirements will generally be able to meet a number of performance criteria, when subjected to earthquake ground motions of differing severity. The performance criteria discussed here were jointly developed during the BSSC Guidelines and Commentary for Seismic Rehabilitation of Buildings Project (ATC, 1995) and the Structural Engineers Association of California *Vision 2000* Project (SEAOC, 1995). In the system established by these projects, earthquake performance of *structures* is defined in terms of several standardized performance levels and reference ground motion levels. Each performance level is defined by a limiting state in which specified levels of degradation and damage have occurred to the structural and nonstructural building *components.* The ground motion levels are defined in terms of their probability of exceedance.

Four performance levels are commonly described as meaningful for the design of *structures.* Although other terminology has been used in some documents, these may respectively be termed the operational, immediate occupancy, life safety, and collapse prevention levels. Of these, the operational level represents the least level of damage to the *structure. Structures* meeting this level when responding to an earthquake are expected to experience only negligible damage to their structural systems and minor damage to nonstructural systems. The *structure* will retain nearly all of its pre-earthquake strength and stiffness and all mechanical, electrical, plumbing, and other systems necessary for the normal operation of the *structure* are expected to be functional. If repairs are required, these can be conducted at the convenience of the occupants. The risk to life safety during an earthquake in a *structure* meeting this performance level is negligible. Note, that in order for a *structure* to meet this level, all utilities required for normal operation must be available, either through standard public service or emergency sources maintained for that purpose. Except for very low levels of ground motion, it is generally not practical to design *structures* to meet this performance level.

The immediate occupancy level is similar to the operational level although somewhat more damage to non-structural systems is anticipated. Damage to the structural systems is very slight and the *structure* retains all of its pre-earthquake strength and nearly all of its stiffness. Nonstructural elements, including ceilings and cladding, but also mechanical and electrical *components*, remain secured and do not represent hazards. Exterior nonstructural wall elements and roof elements continue to provide a weather barrier, and be otherwise serviceable. The *structure* remains safe to occupy, however, some repair and clean-up is probably required before the *structure* can be restored to normal service. In particular, it is expected that utilities necessary for normal function of all systems will not be available, although those necessary for life safety systems would be provided. Some equipment and systems used in normal function of the *structure* may experience internal damage due to shaking of the *structure*, but most would be expected to operate if the necessary utility service was available. Similar to the operational level, the risk to life safety during an earthquake in a *structure* meeting this performance level is negligible. Structural repair may be completed at the occupants convenience, however, significant nonstructural repair and cleanup is probably required before normal function of the *structure* can be restored.

At the life safety level, significant structural and nonstructural damage has occurred. The *structure* may have lost a substantial amount of its original lateral stiffness and strength but still retains a significant margin against collapse. The *structure* may have permanent lateral offset and some elements of the *seismic-force resisting system* may exhibit substantial cracking, spalling, yielding and buckling. Nonstructural elements of the *structure,* while secured and not presenting falling hazards, are severely damaged and can not function. The *structure* is not safe for continued occupancy until repairs are instituted as strong ground motion from aftershocks could result in life threatening damage. Repair of the *structure* is expected to be feasible, however, it may not be economically attractive to do so. The risk to life during an earthquake, in a *structure* meeting this performance level is very low.

At the near collapse level a *structure* has sustained nearly complete damage. The *seismic-force resisting system* has lost most of its original stiffness and strength and little margin remains against collapse. Substantial degradation of the structural elements has occurred including extensive cracking and spalling of masonry and concrete elements and buckling and fracture of steel elements. The *structure* may have significant permanent lateral offset. Nonstructural elements of the *structure* have experienced substantial damage and may have become dislodged creating falling hazards. The *structure* is unsafe for occupancy as even relatively moderate ground motion from aftershocks could induce collapse. Repair of the *structure* and restoration to service is probably not practically achievable.

The design ground motion contained in the *Provisions* is taken as two-thirds of the *maximum considered earthquake ground motion*. Such ground motion may have a return period varying from a few hundred years to a few thousand years, depending on the regional seismicity. It is expected that *structures* designed in accordance with the requirements for Group I would achieve the life safety or better performance level for these ground motions. *Structures* designed in accordance with the requirements for Group III should be able to achieve the Immediate Occupancy or better performance level for this ground motion. *Structures* designed to the requirements for Group II would be expected to achieve performance better than the life safety level but perhaps less than the immediate occupancy level for this ground motion.

While the design ground motion represents a rare earthquake event, it may not be the most severe event that could ever effect a site. In zones of moderate seismicity, it has been common practice in the past to consider ground motion with a 98 percent chance of non-exceedance in 50 years, or an average return period of 2,500 years, as being reasonably representative of the most severe ground motion ever likely to effect a site. This earthquake has been variously termed a maximum credible earthquake, maximum capable event and, most recently, a *maximum considered earthquake*. The recent terminology is adopted here in recognition that ground motion of this probability level is not the most severe motion that could ever effect the site, but is considered sufficiently improbable that more severe ground motions need not practically be considered. In regions near major active faults, such as coastal California, estimates of ground motion at this probability of exceedance can produce structural demands much larger than has typically been recorded in past earthquakes. Consequently, in these zones, the *maximum considered earthquake* is now commonly taken based on conservative estimates of the ground motion from a deterministic event, representing the largest magnitude event that the nearby faults are believed capable of producing.

It is expected that *structures* designed to the requirements for Group I would be capable of responding to the *maximum considered earthquake* at a near collapse or better performance level. *Structures* designed to the requirements for Group III should be capable of responding to such ground motions at the life safety level. *Structures* designed and constructed to the requirements for Group II *structures* should be capable of responding to *maximum considered earthquake ground motions* with a performance intermediate to the near collapse and life safety levels.

In zones of high seismicity, *structures* may experience strong motion earthquakes several times during their lives. It is also important to consider the performance expected of *structures* for these somewhat less severe, but much more frequent, events. For this purpose, earthquake ground shaking with a 50 percent probability of non-exceedance in 50 years may be considered. Sometimes termed a maximum probable event (MPE), such ground motion would be expected to recur at a site, one time, every 72 years. *Structures* designed to the requirements for Group I would be expected to respond to such ground motion at the Immediate Occupancy level. *Structures* designed and constructed to either the Group II or Group III requirements would be expected to perform to the Operational level for these events. This performance is summarized in Figure C1.3.

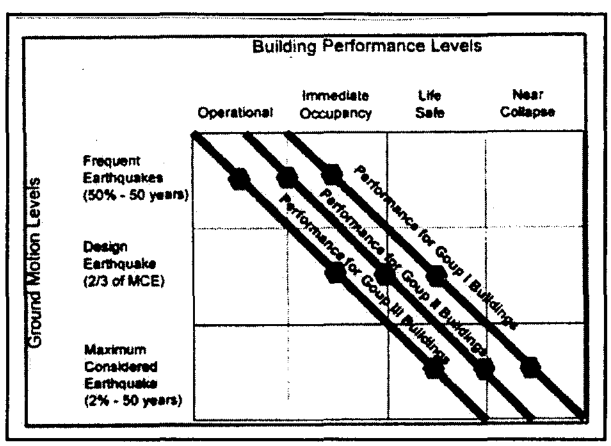


Figure C1.3 Expected building performance

It is important to note that while the performance indicated in Figure C1.3 is generally indicative of that expected for *structures* designed in accordance with the *Provisions*, there can be significant variation in the performance of individual *structures* from these expectations. This variation results from individual site conditions, quality of construction, structural systems, detailing, overall configuration of the *structure*, inaccuracies in our analytical techniques and a number of other complex factors. As a result of these many factors, and intentional conservatism contained in the *Provisions*, most *structures* will perform better than indicated in the figure and others will not perform as well.

1.3.5 Seismic Use Group III Structure Access Protection: This section establishes the requirement for access protection for *Seismic Use Group* III *structures*. There is a need for ingress/egress to those *structures* that are essential post-earthquake facilities and this shall be considered in the siting and design of the *structure*.

1.4 OCCUPANCY IMPORTANCE FACTOR: The concept of an *occupancy importance factor* for structural systems has been included in the *Uniform Building Code* for many years, however, it was first adopted into the 1997 Edition of the *Provisions.* The inclusion of the *occupancy importance factor* is one of several requirements included in this edition of the *Provisions* where there are attempts to control the seismic performance capability of *structures* in the different *Seismic Use Groups*. Specifically, the *occupancy importance factor* modifies the *R* coefficients used to determine minimum design *base shear* forces. *Structures* assigned *occupancy importance factors* greater than 1.0 must be designed for larger *base shear* forces. As a result, these *structures* are expected to experience lower ductility demands than *structures* designed with lower *occupancy importance factors* and, hence, these *structures* would be expected to sustain less damage. The *Provisions* also include requirements that attempt to limit vulnerability to structural damage by specifying more stringent drift limits for *structures* in *Seismic Use Groups* of higher risk. Further discussion of these concepts is found in *Commentary* Sec. 5.2. and 5.2.8.

Chapter 2 Commentary

GLOSSARY AND NOTATIONS

2.1 GLOSSARY:

Active Fault: A fault for which there is an average historic slip rate of 1mm per year or more and geographic evidence of seismic activity within Holocene times (past 11,000 years).

Addition: An increase in the *building* area, aggregate floor area, height, or number of stories of a *structure*.

Adjusted Resistance (*D'*): The reference resistance adjusted to include the effects of all applicable adjustment factors resulting from end use and other modifying factors. Time effect factor (λ) adjustments are not included.

Alteration: Any construction or renovation to an existing *structure* other than an *addition*.

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

Approval: The written acceptance by the authority having jurisdiction of documentation that establishes the qualification of a material, system, *component*, procedure, or person to fulfill the requirements of the *Provisions* for the intended use.

Architectural Component Support: Those structural members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between architectural systems, *components*, or elements and the *structure*.

Attachments: Means by which *components* and their supports are secured and connected to the *seismic-force-resisting system* of the structure. Such *attachments* include anchor bolts, welded connections, and mechanical fasteners.

Base: The level at which the horizontal seismic ground motions are considered to be imparted to the *structure*.

Base Shear: Total design lateral force or shear at the *base*.

Basement: A *basement* is any story below the lowest *story* above grade.

Boundary Elements: *Diaphragm* and *shear wall boundary members* to which sheathing transfers forces. *Boundary members* include chords and drag *struts* at *diaphragm* and *shear wall* perimeters, interior openings, discontinuities, and re-entrant corners.

Braced Wall Line: A series of *braced wall panels* in a single *story* that meets the requirements of Sec. 12.5.2.

Braced Wall Panel: A section of *wall* braced in accordance with Sec. 12.5.2.

Building: Any *structure* whose use could include shelter of human occupants.

Boundary Members: Portions along *wall* and *diaphragm* edges strengthened by longitudinal and transverse reinforcement and/or structural steel members.

Cantilevered Column System: A *seismic-force-resisting system* in which lateral forces are resisted entirely by columns acting as cantilevers from the foundation.

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

> **Component, Equipment:** A mechanical or electrical *component* or element that is part of a mechanical and/or electrical system within or without a *building* system.
>
> **Component, Flexible:** *Component,* including its *attachments,* having a fundamental period greater than 0.06 sec.
>
> **Component, Rigid:** *Component,* including its *attachments,* having a fundamental period less than or equal to 0.06 sec.

Concrete:

> **Plain Concrete:** *Concrete* that is either unreinforced or contains less reinforcement than the minimum amount specified in ACI 318 for *reinforced concrete.*
>
> **Reinforced Concrete:** *Concrete* reinforced with no less than the minimum amount required by ACI 318, prestressed or non-prestressed, and designed on the assumption that the two materials act together in resisting forces.

Confined Region: The portion of *reinforced concrete component* in which the concrete is confined by closely spaced *special transverse reinforcement* restraining the concrete in directions perpendicular to the applied stress.

Construction Documents: The written, graphic, electronic, and pictorial documents describing the design, locations, and physical characteristics of the project required to verify compliance with the *Provisions*.

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

Coupling Beam: A beam that is used to connect adjacent concrete *wall* piers to make them act together as a unit to resist lateral loads.

Damping Device: A flexible structural element of the *damping* system that dissipates energy due to relative motion of each end of the device. *Damping devices* include all pins, bolts gusset plates, brace extensions, and other components required to connect damping devices to the other elements of the *structure. Damping devices* may be classified as either displacement-dependent or velocity-dependent, or a combination thereof, and may be configured to act in either a linear or nonlinear manner.

Damping System: The collection of structural elements that includes all the individual *damping devices,* all structural elements or bracing required to transfer forces from *damping devices* to the base of the *structure,* and the structural elements required to transfer forces from damping *devices* to the *seismic-force-resisting system.*

Deformability: The ratio of the ultimate *deformation* to the limit *deformation.*

> **High Deformability Element:** An element whose *deformability* is not less than 3.5 when subjected to four fully reversed cycles at the limit *deformation.*
>
> **Limited Deformability Element:** An element that is neither a low *deformability* nor a high deformability element.

Low Deformability Element: An element whose *deformability* is 1.5 or less.

Deformation:

> **Limit Deformation:** Two times the initial *deformation* that occurs at a load equal to 40 percent of the maximum strength.
>
> **Ultimate Deformation:** The *deformation* at which failure occurs and which shall be deemed to occur if sustainable load reduces to 80 percent or less of the maximum strength.

Design Earthquake Ground Motion: The earthquake effects that *buildings* and *structures* are specifically proportioned to resist as defined in Sec. 4.1.

Design Earthquake: Earthquake effects that are two-thirds of the corresponding *maximum considered earthquake.*

Designated Seismic System: Those architectural, electrical, and mechanical systems and their *components* that require design in accordance with Sec. 6.1 and that have a *component* importance factor (I_p) greater than 1.

Diaphragm: A roof, floor, or other membrane system acting to transfer lateral forces to the vertical resisting elements. *Diaphragms* are classified as either flexible or rigid according to the requirements of Sec. 5.2.3.1 and 12.4.1.1.

Diaphragm, Blocked: A *diaphragm* in which all sheathing edges not occurring on a framing member are supported on and fastened to blocking.

Diaphragm Boundary: A location where shear is transferred into or out of the *diaphragm* sheathing. Transfer is either to a *boundary element* or to another free-resisting element.

Diaphragm Cord: A *diaphragm boundary element* perpendicular to the applied load that is assumed to take axial stresses due to the *diaphragm* moment in a manner analogous to the flanges of a beam. Also applies to *shear walls.*

Displacement:

> **Design Displacement:** The *design earthquake* lateral *displacement,* excluding additional *displacement* due to actual and accidental torsion, required for design of the *isolation system.*

Total Design Displacement: The *design earthquake* lateral *displacement,* including additional *displacement* due to actual and accidental torsion, required for design of the *isolation system* or an element thereof.

Total Maximum Displacement: The *maximum considered earthquake* lateral *displacement*, including additional *displacement* due to actual and accidental torsion, required for verification of the stability of the *isolation system* or elements thereof, design of *structure* separations, and vertical load testing of *isolator unit* prototypes.

Displacement-Dependent Damping Device: The force response of a *displacement-dependent damping device* is primarily a function of the relative displacement between each end of the device. The response is substantially independent of the relative velocity between each end of the device and/or the excitation frequency.

Displacement Restraint System: A collection of structural elements that limits lateral *displacement* of seismically isolated structures due to *maximum* considered *earthquake* ground shaking.

Drag Strut (Collector, Tie, Diaphragm Strut): A *diaphragm* or *shear wall boundary element* parallel to the applied load that collects the transfered *diaphragm* shear forces to the vertical-force-resisting elements or distributes forces within the *diaphragm or shear wall.* A *drag strut* often is an extension of a *boundary element* that transfers forces into the *diaphragm or shear wall.*

Effective Damping: The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the *isolation system*.

Effective Stiffness: The value of lateral force in the *isolation system*, or an element thereof, divided by the corresponding lateral *displacement.*

Enclosure: An interior space surrounded by *walls.*

Equipment Support: Those structural members or assemblies of members or manufactured elements, including braces, frames, legs, lugs, snuggers, hangers or saddles, that transmit *gravity load* and operating load between the equipment and the *structure.*

Essential Facility: A facility or structure required for post-earthquake recovery.

Factored Resistance ($\lambda\phi$D): *Reference resistance* multiplied by the time effect and resistance factors. This value must be adjusted for other factors such as size effects, moisture conditions, and other end-use factors.

Flexible Equipment Connections: Those connections between equipment *components* that permit rotational and/or transitional movement without degradation of performance. Examples included universal joints, bellows expansion joints, and flexible metal hose.

Frame:

Braced Frame: An essentially vertical truss, or its equivalent, of the concentric or eccentric type that is provided in a *building frame system* or *dual frame system* to resist shear.

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

Eccentrically Braced Frame (EBF): A diagonally *braced frame* in which at least one end of each brace frames into a beam a short distance from a beam-column joint or from another diagonal brace.

Ordinary Concentrically Braced Frame (OCBF): A steel *concentrically braced frame* in which members and connections are designed in accordance with the provisions of AISC Seismic without modification.

Special Concentrically Braced Frame (SCBF): A steel or composite steel and concrete *concentrically braced frame* in which members and connections are designed for ductile behavior

Moment Frame: A frame provided with restrained connections between the beams and columns to permit the frame to resist lateral forces through the flexural rigidity and strength of its members.

Intermediate Moment Frame: A *moment frame* of reinforced concrete meeting the detailing requirements of ACI 318, of structural steel meeting the detailing requirements of AISC Seismic, or of composite construction meeting the requirements of AISC Seismic.

Ordinary Moment Frame: A *moment frame* or reinforced concrete conforming to the requirements of ACI 318 exclusive of Chapter 21, of structural steel meeting the detailing requirements of AISC Seismic or of composite construction meeting the requirements of AISC Seismic

Special Moment Frame: A *moment frame* of reinforced concrete meeting the detailing requirements of ACI 318, of structural steel meeting the detailing requirements of AISC Seismic, or of composite construction meeting the requirements of AISC Seismic.

Frame System:

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

Dual Frame System: A structural system with an essentially complete *space frame system* providing support for vertical loads. Seismic force resistance is provided by a *moment resisting frame* and *shear walls* or *braced frames* as prescribed in Sec. 5.2.2.1

Space Frame System: A structural system composed of interconnected members, other than *bearing walls*, that is capable of supporting vertical loads and that also may provide resistance to shear.

Glazed Curtain Wall: A *nonbearing wall* that extends beyond the edges of the building floor slabs and includes a glazing material installed in the curtain wall framing.

Glazed Storefront: A *nonbearing wall* that is installed between floor slabs typically including entrances and includes a glazing material installed in the storefront framing.

Grade Plane: A reference plane representing the average of the finished ground level adjoining the *structure* at the exterior *walls.* Where the finished ground level slopes away from the exterior walls, the reference plane shall be established by the lowest points within the area between the *buildings* and the lot line or, where the lot line is more than 6 ft (1829 mm) from the *structure,* between the *structure* and a point 6 ft (1829 mm) from the *structure.*

Hazardous Contents: A material that is highly toxic or potentially explosive and in sufficient quantity to pose a significant life-safety threat to the general public if an uncontrolled release were to occur.

High Temperature Energy Source: A fluid, gas, or vapor whose temperature exceeds 220 degrees F (378 K).

Inspection, Special: The observation of the work by the *special inspector* to determine compliance with the approved *construction documents* and the *Provisions.*

> **Continuous Special Inspection:** A full-time observation of the work by an approved *special inspector* who is present in the area where work is being performed.
>
> **Periodic Special Inspection:** The part-time or intermittent observation of the work by an approved *special inspector* who is present in the area where work has been or is being performed.

Inspector, Special (who shall be identified as the Owner's Inspector): A person approved by the authority having jurisdiction as being qualified to perform *special inspection* required by the approved *quality assurance plan.* The quality assurance personnel of a fabricator is permitted to be approved by the authority having jurisdiction as a *special inspector.*

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

Isolation Interface: The boundary between the upper portion of the *structure,* which is isolated, and the lower portion of the *structure,* which moves rigidly with the ground.

Isolation System: The collection of structural elements that includes all individual *isolator units,* all structural elements that transfer force between elements of the *isolation system,* and all connections to other structural elements. The *isolation system* also includes the *wind-restraint system,* energy-dissipation devices, and/or the *displacement restraint system* if such systems and devices are used to meet the design the requirements of Chapter 13.

Isolator Unit: A horizontally flexible and vertically stiff structural element of the *isolation system* that permits large lateral *deformations* under design seismic load. An *isolator unit* is permitted to be used either as part of or in addition to the weight-supporting system of the *structure.*

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

Load:

> **Dead Load:** The *gravity load* due to the weight of all permanent structural and nonstructural *components* of a *building* such as *walls,* floors, roofs, and the operating weight of fixed service equipment.
>
> **Gravity Load (W):** The total *dead load* and applicable portions of other loads as defined in Sec. 5.4.1.
>
> **Live Load:** The load superimposed by the use and occupancy of the *building* not including the wind load, earthquake load, or *dead load*; see Sec. 5.4.1.

Maximum Considered Earthquake Ground Motion: The most severe earthquake effects considered by the *Provisions* as defined in Sec. 4.1.

Nonbuilding Structure: A *structure,* other than a *building*, constructed of a type included in Chapter 14 and within the limits of Sec. 14.1.1.

Occupancy Importance Factor: A factor assigned to each *structure* according to its *Seismic Use Group* as prescribed in Sec. 1.4.

Owner: Any person, agent, firm, or corporation having a legal or equitable interest in the property.

Partition: A nonstructural interior *wall* that spans from floor to ceiling, to the floor or roof structure immediately above, or to subsidiary structural members attached to the *structure* above.

P-Delta Effect: The secondary effect on shears and moments of structural members induced due to *displacement* of the *structure.*

Quality Assurance Plan: A detailed written procedure that establishes the systems and *components* subject to *special inspection* and testing.

Reference Resistence: The resistence (force or moment as appropriate) of a member or connection computed at the reference end use conditions.

Registered Design Professional: An architect or engineer registered or licensed to practice professional architecture or engineering as defined by statuary requirements of the professional registrations laws of the state in which the project is to be constructed.

Roofing Unit: A unit of roofing material weighing more than 1 pound (0.5 kg).

Seismic Design Category: A classification assigned to a *structure* based on its *Seismic Use Group* and the severity of the *design earthquake ground motion* at the site.

Seismic-Force-Resisting System: That part of the structural system that has been considered in the design to provide the required resistence to the *shear wall* prescribed herein.

Seismic Forces: The assumed forces prescribed herein, related to the response of the *structure* to earthquake motions, to be used in the design of the *structure* and its *components.*

Seismic Response Coefficient: Coefficient C_s as determined from Sec. 5.4.1.

Seismic Use Group: A classification assigned to the *structure* based on its use as defined in Sec. 1.3.

Shallow Anchors: Anchors with embedment length-to-diameter ratios of less than 8.

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

Site Class: A classification assigned to a site based on the types of soils present and their engineering as defined in Sec. 4.1.2.

Site Coefficients: The values of F_a and F_v indicated in Tables 4.1.2.4a and 4.1.2.4b, respectively.

Special Transverse Reinforcement: Reinforcement composed of spirals, closed stirrups, or hoops and supplementary cross-ties provided to restrain the concrete and qualify the portion of the *component,* where used, as a confined region.

Storage Racks: Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed and hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

Story: The portion of a *structure* between the top to top of two successive finished floor surfaces and, for the topmost story. From the top of the floor finish to the top of the roof structural element.

Story Above Grade: Any *story* having its finished floor surface entirely above grade, except that a *story* shall be considered as the *story above grade* where the finished floor surface of the *story* immediately above is more than 6 ft (1829 mm) above the *grade plane,* more than 6 ft (1829 mm) above the finished ground level for more than 40 percent of the total *structure* perimeter, or more than 12 ft (3658 mm) above the finished ground level at any point. This definition is illustrated in Figure 2.1.

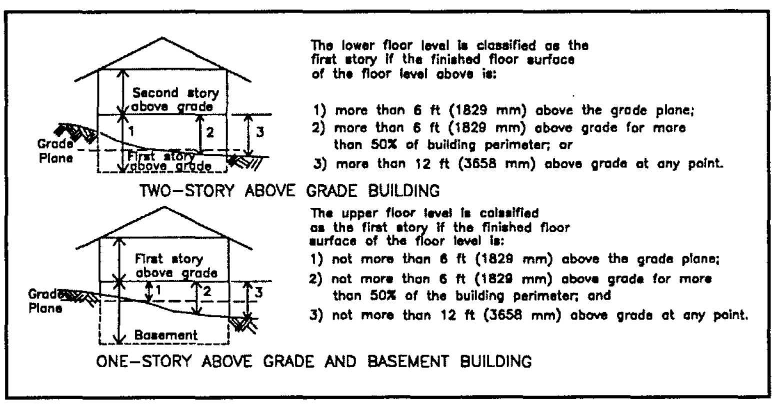


FIGURE 2.1 Definition of story above grade.

Story Drift Ratio: The *story* drift, as determined in Sec. 5.4.6, divided by the *story* height.

Story Shear: The summation of design lateral forces at levels above the *story* under consideration.

Strength:

Design Strength: *Nominal strength* multiplied by the strength reduction factor, ϕ.

Nominal Strength: Strength of a member or cross section calculated in accordance with the requirements and assumptions of the strength design methods of the *Provisions* (or the reference standards) before application of any strength reduction factors.

Required Strength: Strength of a member, cross section, or connection required to resist factored loads or related internal moments and forces in such combinations as stipulated by the *Provisions.*

Structure: That which is built or constructed and limited to *buildings* and *nonbuilding structures* as defined herein.

Structural Observations: The visual observations performed by the *registered design professional* in responsible charge (or another *registered design professional)* to determine that the *seismic-force-resisting system* is constructed in general conformance with the *construction documents.*

Wood Structural Panel: A wood-based panel product that meets the requirements of PS 1 or PS 2 and is bonded with a waterproof adhesive. Included under this designation is plywood, oriented strand board, and composite panels.

Subdiaphragm: A portion of a diaphragm used to transfer *wall* anchorage forces to the *diaphragm* cross ties.

Testing Agency: A company or corporation that provides testing and/or inspection services. The person in responsible charge of the *special inspector(s)* and the testing services shall be a *registered design professional.*

Tie-Down (Hold-Down): A device used to resist uplift of the chords of *shear walls*. These devices are intended to resist load without significant slip between the device and the *shear wall* chord or be shown with cyclic testing to not reduce the *wall* capacity and ductility.

Time Effect Factor: A factor applied to the adjusted resistence to account for effects of duration load.

Torsional Force Distribution: The distribution of horizontal *shear wall* through the rigid *diaphragm* when the center of the mass of the *structure* at the level under consideration does not coincide with the center of the rigidity (sometimes referred to as diaphragm rotation).

Toughness: The ability of a material to absorb energy without losing significant *strength.*

Utility or Service Interface: The connection of the *structure's* mechanical and electrical distribution systems to the utility or service company's distribution system.

Velocity-Dependent Damping Device: The force-displacement relation for a *velocity-dependent damping device* is primarily a function of the relative velocity between each end of the device and also may be a function of the relative displacement between each end of the device.

Veneers: Facings or ornamentations of brick, concrete, stone, tile, or similar materials attached to a backing.

Wall: A *component* that has a slope of 60 degrees or greater with the horizontal plane used to enclose or divide space.

Bearing Wall: An exterior or interior *wall* providing support for vertical loads.

Cripple Wall: A framed stud *wall,* less than 8 ft (2400 mm) in height, extending from the top of the foundation to the underside of the lowest floor framing. *Cripple walls* can occur in both engineered *structures* and conventional construction.

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

Light-Framed Wood Shear Wall: A *wall* constructed with wood studs and sheathed with material rated for shear resistance.

Nonbearing Wall: An exterior or interior *wall* that does not provide support for vertical loads other than its own weight or as permitted by the building code administered by the authority having jurisdiction.

Nonstructural Wall: All walls other than *bearing walls* or *shear walls.*

Shear Wall (Vertical Diaphragm): A *wall* designed to resist lateral forces parallel to the plane of the *wall* (sometimes referred to as a vertical *diaphragm).*

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

Wind-Restraint System: The collection of structural elements that provides restraint of the seismic-isolated *structure* for wind loads. The *wind-restraint system* may be either an integral part of *isolator units* or a separate device.

2.2 NOTATIONS:

A, B, C, D, E, F	*Site classes* as defined in Sec. 4.1.2.
A_b	Area (in.^{2} or mm^2) of anchor bolt or stud in Chapters 6 and 11.
A_{ch}	Cross sectional-area (in.^{2} or mm^2) of a *component* measured to the outside of the special lateral reinforcement.
A_n	Net-cross sectional area of masonry (in.^{2} or mm^2) in Chapter 11.
A_o	The area of the load-carrying foundation (ft^2 or m^2).
A_p	Projected area on the masonry surface of a right circular cone for anchor bolt allowable shear and tension calculations (in.^{2} or mm^2) in Chapter 11.
A_s	The area of an assumed failure surface taken as a pyramid in Eq. 2.4.1-3 or in Chapter 9.
A_s	Cross-sectional area of reinforcement (in.^{2} or mm^2) in Chapters 6 and 11.

A_{sh}	Total cross-sectional area of hoop reinforcement (in.^{2} or mm^2), including supplementary cross-ties, having spacing of s_h and crossing a section with a core dimension of h_c.
A_{vd}	Required area of leg (in.^{2} or mm^2) of diagonal reinforcement.
A_x	The torsional amplification factor.
a_b	Length of compressive stress block (in. or mm) in Chapter 11.
a_d	The incremental factor related to *P-delta effects* in Sec. 5.4.5.
a_p	The *component* amplification factor as defined in Sec. 6.1.3.
B_a	Nominal axial strength of an anchor bolt (lb or N) in Chapter 11.
B_D	Numerical coefficient as set forth in Table 13.3.3.1 for effective damping equal to β_D.
B_{ID}	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to β_{ml} (m=1) and period of *structure* equal to T_{1I}.
B_m	Numerical coefficient as set forth in Table 13.3.3.1 for effective damping equal β_M
B_{IM}	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to β_{mM} (m=1) and period of *structure* equal to T_{IM}.
B_{mD}	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to β_{ml} and period of *structure* equal to T_m.
B_{mM}	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to β_{mM} and period of *structure* equal to T_m.
B_R	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to β_R and the period of *structure* equal to T_R.
B_v	Nominal shear strength of an anchor bolt (lb or N) in Chapter 11.
$B_{V\text{-}I}$	Numerical coefficient as set forth in Table 13A.3.1 for effective damping equal to the sum of viscous damping in the fundamental mode of vibration of the *structure* in the direction of interest, β_{Vm} (m = 1), plus inherent damping, β_I, and period of *structure* equal to T_1.
b	The shortest plan dimension of the *structure,* in ft (mm), measured perpendicular to d.
b_a	Factored axial force on an anchor bolt (lb or N) in Chapter 11.
b_v	Factored shear force on an anchor bolt (lb or N) in Chapter 11.
b_w	Web width (in.or mm) in Chapter 11.

C_u	Coefficient for upper limit on calculated period; see Table 5.4.2.
C_d	The deflection amplification factor as given in Table 5.2.2.
C_{mFD}	Force coefficient as set forth in Table 13A.7.3.2.1.
C_{mFV}	Force coefficient as set forth in Table 13A.7.3.2.2.
C_s	The *seismic response coefficient* (dimension-less) determined in Sec. 5.4.1.1.
C_{S1}	Seismic response coefficient (dimension-less) of the fundamental mode of vibration of the *structure* in the direction of interest. Sec. 13A.4.3.4 or Sec. 13A.5.3.4 (m = 1).
C_{sm}	The modal *seismic response coefficient* (dimension-less) determined in Sec. 5.5.4..
C_{Sm}	*Seismic response coefficient* (dimension-less) of the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 13A.5.3.4 (m = 1) or Sec. 13A.5.3.6 (m > 1).
C_{SR}	*Seismic response coefficient* (dimension-less) of the residual mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.8.
C_{vx}	The vertical distribution factor as determined in Sec. 5.4.3.
c	Distance from the neutral axis of a flexural member to the fiber of maximum compressive strain (in. or mm).
c_{eq}	Effective energy dissipation device damping coefficient (Eq. 13.3.2.1).
D	Reference resistance in Chapter 12.
D	The effect of *dead load* in Sec. 5.2.7 and Chapter 13.
D	Adjusted resistance in Chapter 12.
D_D	*Design displacement* (in. or mm) at the center of rigidity of the *isolation system* in the direction under consideration as prescribed by Eq. 13.3.3.1.
D_D'	*Design displacement* (in. or mm), at the center of rigidity of the *isolation system* in the direction under consideration, as prescribed by Eq. 13.4.2-1.
D_{1D}	Fundamental mode *design displacement* at the center rigidity of the roof level of *structure* in the direction under consideration, Sec. 13A.4.4.3 (in. or mm).
D_{1M}	Fundamental mode *maximum displacement* at the center of rigidity of the roof level of the *structure* in the direction under consideration, Sec. 13A.4.4.6 (in. or mm).

D_{mD} — *Design displacement* at the center of rigidity of the roof level of the *structure* due to the m^{th} mode of vibration in the direction under consideration, Sec. 13A.5.4.3 (in. or mm).

D_{mM} — *Maximum displacement* at the center of rigidity of the roof level of the *structure* due to the m^{th} mode of vibration in the direction under consideration, Sec. 13A.5.4.6 (in. or mm).

D_M — Maximum *displacement* (in. or mm), at the center of rigidity of the *isolation system* in the direction under consideration as prescribed by Eq. 13.3.3.3.

D_M — *Maximum displacement* (in. or mm), at the center of rigidity of the *isolation system* in the direction under consideration as prescribed by Eq. 13.4.2-2.

D_p — Relative seismic *displacement* that the *component* must be designed to accommodate as defined in Sec. 6.1.4.

D_{RD} — Residual mode *design displacement* at the center of rigidity of the roof level of the *structure* in the direction under consideration, Sec. 13A.4.4.3 (in. or mm).

D_{RM} — Residual mode *maximum displacement* at the center of rigidity of the roof level of the *structure* in the direction under consideration, Sec. 13A.4.4.6 (in. or mm).

D_s — The total depth of the stratum in Eq. 5.8.2.1.2-4 (ft or m).

D_Y — Displacement at the center of rigidity of the roof level of the *structure* at the effective yield point of the *seismic-force-resisting system,* Sec. 13A.3.4 (in. or mm).

D_{TD} — Total *design displacement* (in. or mm), of an element of the *isolation system* including both translational *displacement* at the center of rigidity and the *component* of torsional *displacement* in the direction under consideration as prescribed by Eq. 13.3.3.5-1.

D_{TM} — Total *maximum displacement* (in. or mm), of an element of the *isolation system* including both translational *displacement* at the center of rigidity and the component of torsional *displacement* in the direction under consideration as prescribed by Eq. 13.3.3.5-2.

d — Overall depth of member (in.or mm) in Chapters 5 and 11.

d — The longest plan dimension of the *structure* (ft.or mm) in Chapter 13.

d_b — Diameter of reinforcement (in.or mm) in Chapter 11.

d_e — Distance from the anchor axis to the free edge (in.or mm) in Chapter 9.

d_p — The longest plan dimension of the *structure* (ft or mm).

E	The effect of horizontal and vertical earthquake-induced forces (Sec. 5.2.7 and Chapter 13).
E_{loop}	Energy dissipated (kip-inches or kN-mm), in an *isolator unit* during a full cycle of reversible load over a test *displacement* range from Δ+ to Δ- as measured by the area enclosed by the loop of the loop of the force-deflation curve.
E_m	Chord modulus of elasticity of masonry (psi or MPa) in Chapter 11.
E_s	Modulus of elasticity of reinforcement (psi or MPa) in Chapter 11.
E_v	Modulus of rigidity of masonry (psi or Mpa) in Chapter 11.
e	The actual eccentricity (ft or mm), measured in plan between the center of mass of the *structure* above the isolation interface and the center of rigidity of the *isolation system,* plus accidental eccentricity (ft or mm), taken as 5 percent the maximum *building* dimension perpendicular to the direction of the force under consideration.
F_a	Acceleration-based site coefficient (at 0.3 sec period).
F-	Maximum negative force in an *isolator unit* during a single cycle of prototype testing a *displacement* amplitude of Δ-.
F-	Positive force in kips (kN) in an *isolator unit* during a single cycle of prototype testing at a *displacement* amplitude of Δ-.
F_i, F_n, F_x	The portion of the seismic base shear, V, induced at level i, n, or x, respectively, as determined in Sec. 5.4 (kip or kN).
F_{il}	Inertial force at Level i (or mass point i) in the fundamental mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.9.
F_{im}	Inertial force at Level i (or mass point i) in the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 13A.5.3.7.
F_{iR}	Inertial force at Level i (or mass point i) in the residual mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.9.
F_p	The seismic design force center of gravity and distributed relative to the *component's* weight distribution as determined in Sec. 6.1.3.
F_v	Velocity-based site coefficient (at 1.0 sec period).
F_x	Total force distributed over the height of the *structure* above the isolation interface as prescribed by Eq. 13.3.5.
F_{xm}	The portion of the seismic *base shear,* V_m, induced at a Level x as determined in Sec. 5.5.5 (kip or kN).
f_c'	Specified compressive strength of concrete used in design

f_i	Lateral force at Level *i* of the *structure* distributed approximately in accordance with Equation 5.3.4-2, Sec. 13A.4.3.3.
f_m'	Specified compressive strength of masonry (psi or MPa) at the age of 28 days unless a different age is specified, Chapter 11.
f_r	Modulus of rupture of masonry (psi or MPa) in Chapter 11.
f_s'	Ultimate tensile strength (psi or MPa) of the bolt, stud or insert leg wires. For A307 bolts or A108 studs, is permitted to be assumed to be 60,000 psi (415 Mpa).
f_y	Specified yield strength of reinforcement (psi or kPa).
f_{yh}	Specified yield stress of the special lateral reinforcement (psi or kPa).
G	yv_s^2/g = the average shear modulus for the soils beneath the foundation at large strain levels (psf of Pa).
G_o	yv_{so}^2/g = the average shear modulus for the soils beneath the foundation at small strain levels (psf of Pa).
g	Acceleration of gravity in in./sec^2 (mm/s^2).
H	Thickness of soil.
h	The height of a *shear wall* measured as the maximum clear height from the foundation to the bottom of the floor or roof framing above or the maximum clear height from the top of the floor or roof framing to the bottom of the floor or roof framing above.
$\overline{h}$	The effective height of the *building* as determined in Sec. 5.8.2.1.1 (ft or m).
h	Height of a wood shear panel or *diaphragm* (ft or mm) in Chapter 12.
h	The roof elevation of a *structure* in Chapter 6.
h	Height of the member between points of support (in. or mm) on Chapter 11.
h_c	The core dimension of a *component* measured to outside of the special lateral reinforcement (in. or mm).
h_i, h_n, h_x	The height above the *base* Level *I*, *n*, or *x*, respectively (ft or m).
h_r	Height of the *structure* above the *base* to the roof level (ft or m), Sec. 13A.4.3.3.
h_{sx}	The *story* height below Level $x = h_x - h_{x-1}$ (ft. Or m).
I	The *occupancy importance factor* in Sec. 1.4.
I_{cr}	Moment of inertia of the cracked section (in.^{4} or mm) in Chapter 11.

I_n	Moment of inertia of the net cross-sectional area of a member (in.^{4} or mm^4) in Chapter 11.
I_o	The static moment of inertia of the load-carrying foundation , see Sec. 5.8.2.1 (in.^{4} or mm^4).
I_p	The *component* importance factor as prescribed in Sec. 6.1.5.
I	The *building* level referred to by the subscript I; $I = 1$ designates the first level above the *base.*
K_p	The stiffness of *component* or attachment as defined in Sec. 6.3.3.
K_y	The lateral stiffness of the foundation as defined in Sec. 5.8.2.1.1 (lb/in. or N/m).
K_θ	The rocking stiffness of the foundation as defined in Sec. 5.8.2.1.1 (ft.lb/degree or N m/rad).
KL/r	The lateral slenderness of a compression member measured in terms of its effective buckling length, KL, and the least radius of gyration of the member cross section, $\boldsymbol{r}$.
k	The distribution exponent given in Sec. 5.4.3.
K_{dmax}	Maximum effective stiffness, in kips/inch (kN/mm) of the *isolation system* at the *design displacement* in the horizontal direction under consideration as prescribed by Eq. 13.9.5.1-1.
K_{Dmin}	Minimum effective stiffness (kips/inch or kN/mm) of the *isolation system* at the *design displacement* in the horizontal direction under consideration as prescribed by Eq. 13.9.5.1-2.
K_{max}	Maximum effective stiffness (kips/inch or kN/mm) of the *isolation system* at the maximum *displacement* in the horizontal direction under consideration as prescribed by Eq 13.9.5.1-3.
K_{Min}	Minimum effective stiffness (kips/inch or kN/mm) of the *isolation system* at the maximum *displacement* in the horizontal direction under consideration, as prescribed by Eq. 13.9.5.1-4.
k_{eff}	Effective stiffness of an *isolator unit* as prescribed by Eq. 13.9.3-1.
$\bar{k}$	The stiffness of the *building* as determined in Sec. 5.8.2.1.1 (lb/ft or N/m).
L	The overall length of the *building* (ft or m) at the *base* in the direction being analyzed.
L	Length of bracing member (in. or mm) in Chapter 8.
L	Length of coupling beam between coupled *shear walls* in Chapter 11 (in. or mm).

L	The effect of *live load* in Chapter 13.
L_o	The overall length of the side of the foundation in the direction being analyzed, Sec. 5.8.2.1.2 (ft or m).
l	The dimension of a *diaphragm* perpendicular to the direction of application of force. For open-front *structures,* l is the length from the edge of the *diaphragm* at the open front to the vertical resisting elements parallel to the direction of the applied force. For a cantilevered *diaphragm,* l is the length from the edge of the *diaphragm* at the open front to the vertical resisting elements parallel to the direction of the applied force.
ℓ_b	Effective embedment length of anchor bolt (in. or mm) in Chapter 11.
ℓ_{bc}	Anchor bolt edge distance (in. or mm) in Chapter 11.
ℓ_d	Development length (in. or mm) in Chapter 11.
ℓ_{dh}	Equivalent development length for a standard hook (in. or mm) in Chapter 11.
ℓ_{ld}	Minimum lap splice length (in. or mm) in Chapter 11.
M	Moment on a masonry section due to un-factored loads (in. lb or N · mm) in Chapter 11.
M_a	Maximum moment in a member at deflation is computed (in. ·lb or N · mm) in Chapter 11.
M_{cr}	Cracking moment strength of the masonry (in. ·lb or N ·mm) in Chapter 11.
M_d	Design moment strength (in. ·lb or N ·mm) in Chapter 11.
M_f	The foundation overturning design moment as defined in Sec. 5.4.5 (ft ·kip or kN ·m).
$M_{o,}$ M_{ol}	The overturning moment at the foundation-soil interface as determined in Sec. 5.8.2 and 5.8.3 (ft ·lb or N ·m)
M_{nb}	Un-factored ultimate moment capacity at balanced strain conditions.
M_t	The torsional moment resulting from the location of the *building* masses (ft ·kip or kN ·m).
M_{ta}	The accidental torsional moment as determined in Sec. 5.4.4.2 (ft ·kip or kN ·m).
M_u	Required flexural strength due to factored loads (in.·lb or N ·mm) in Chapter 11.
M_1, M_2	Nominal moment strength at the ends of the coupling beam (in ·lb or N ·mm) in Chapter 11.

M_x	The *building* overturning design moment at Level x as defined in Sec. 5.4.5 or Sec. 5.5.9 (ft ·kip or kN ·m).
m	A subscript denoting the mode of vibration under consideration; i.e., m=1 for the fundamental mode.
N	Number of *stories*, Sec. 5.4.2.1.
N	Standard penetration resistance, ASTM D1536-84.
$\overline{N}$	Average field standard penetration test for the top 100 ft (30 m); see Sec. 4.1.
N_{ch}	Average standard penetration of cohesion-less soil layers for the top 100 ft (30 m); see Sec. 4.1.
N_v	Force acting normal to shear surface (lb or N) in Chapter 11.
n	Designates the level that is uppermost in the main portion of the *building*.
n	Number of anchors in Chapter 9.
P	Axial load on a masonry section due to unfactored loads (lb or N) in Chapter 11.
P_c	Design tensile strength governed by concrete failure of anchor bolts in Chapter 9.
P_D	Required axial strength on a column resulting from the application of *dead load*, *D,* in Chapter 5 (kip or kN).
P_E	Required axial strength on a column resulting from the application of the amplified earthquake load, E', in Chapter 5 (kip or kN).
P_L	Required axial strength on a column resulting from application of *live load*, *L,* in Chapter 5(kip or kN).
P_n	Nominal axial load strength (lb or N) in Chapter 8.
P_n	The algebraic sum of the *shear wall* and the minimum gravity loads on the joint surface acting simultaneously with the shear (lb or N).
P_n	Nominal axial load strength (lb or N) in Chapter 11.
P_s	Design tensile strength governed by steel of anchor bolts in Chapter 9.
P_u	Required axial load (lb or N) in Chapter 11.
P_u	Tensile strength required due to factored loads (lb or N) in Chapter 9.
P_u^*	Required axial strength on a brace (kip or kN) in Chapter 8.
P_x	The total unfactored vertical design load at and above level x (kip or kN).
PI	Plasticity index, ASTM D4318-93.

Q_{DSD} Force in an element of the *damping system* required to resist design seismic forces of *displacement-dependent damping devices*, Sec. 13A.7.3.2.

Q_E The effect of horizontal seismic forces (kip or kN) in Chapters 5 and 13.

Q_{mDSV} Forces in an element of the *damping system* required to resist design seismic forces of *velocity-dependent damping devices* due to the m^{th} mode of vibration of *structure* in the direction of interest, Sec. 13A.7.3.2.

Q_{mSFRS} Force in a element of the *damping system* equal to the design seismic force of the m^{th} mode of vibration of the *seismic force resisting system* in the direction of interest, 13A.7.3.2.

Q_v The load equivalent to the effect of the horizontal and vertical shear strength of the vertical segment in the Appendix to Chapter 8.

q_H Hysteresis loop adjustment factor as determined in Sec. 13A.3.3.

R The response modification coefficient as given in Table 5.2.2.

R_I Numerical coefficient related to the type of lateral-force-resisting system above the *isolation system* as set forth in Table 13.3.4.2 for seismically isolated *structures*.

R_p The *component* response modification system factor as defined in Chapter 6.

r The characteristic length of the foundation as defined in Chapter 5 (ft or m)

r Radius of gyration (in. or mm) in Chapter 11.

r_a, r_m The characteristic foundation length defined in Sec. 5.8.2.1.1 (ft or m).

r_x The ratio of the design *story shear* resisted by the most heavily loaded single element in the story, in direction x, to the total *story shear.*

S Section modules based on net cross sectional area of a *wall* (in.^{3} or mm^3) in Chapter 11.

S_1 The *maximum considered earthquake,* 5 percent damped, spectral response acceleration at a period of 1 second as defined in Chapter 4.

S_{D1} The design, 5 percent damped, spectral response acceleration at a period of 1 second as defined in Chapter 4..

S_{DS} The design, 5 percent damped, spectral response acceleration at short periods as defined in Chapter 4.

S_{M1} The *maximum considered earthquake,* 5 percent damped, spectral response acceleration at a period of one second adjusted for *site class* effects as defined in Chapter 4.

S_{MS}	The *maximum considered earthquake*, 5 percent damped, spectral response acceleration at short periods adjusted for *site class* effects as defined in Chapter 4.
S_s	The mapped *maximum considered earthquake,* 5 percent damped, spectral response acceleration at short periods as defined in Chapter 4.
S_{pr}	Probable strength of precast element connectors (Sec. 9.1.1.12).
$\overline{s}_u$	Average undrained shear strength in top 100 ft (30.5 m); see Sec. 4.1.2.3, ASTM D2166-91 or ASTM D2850-87.
s_h	Spacing of special lateral reinforcement (in. or mm).
T	The period (sec) of the fundamental mode of vibration of the structure in the direction of interest as determined in Chapter 5.
$\widetilde{T}$, $\widetilde{T}_1$	The effective fundamental period (sec) of the *building* as determined in Chapter 5.
T_1	Period, in seconds, of the fundamental mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.3.
T_a	The approximate fundamental period (sec) of the *building* as determined in Chapter 5.
T_D	Effective period, in seconds (sec), of the seismically isolated *structure* at the design *displacement* in the direction under consideration as prescribed by Eq. 13.3.3.2.
T_{1D}	Effective period, in seconds, of the fundamental mode of vibration of the *structure* at the *design displacement* in the direction under consideration, as prescribed by Sec. 13A.4.3.5 or Sec. 13A.5.3.5.
T_{1M}	Effective period, in seconds, of the fundamental mode of vibration of the *structure* at the *maximum displacement* in the direction under consideration, as prescribed by Sec. 13A.4.3.5 or Sec. 13A.5.3.5.
T_M	Effective period, in seconds (sec), of the seismically isolated *structure* at the maximum *displacement* in the direction under consideration as prescribed by Eq. 13.3.3.4.
T_m	The period (sec) of the m^{th} mode of vibration of the *structure in the direction of interest* determined in Chapter 5.
T_m	Period, in seconds, of the m^{th} mode of vibration of the *structure* in the direction under consideration, Sec. 13A.5.3.6.
T_0	$0.2S_{D1}/S_{DS}$
T_p	The fundamental period (sec) of the *component* and its attachment(s) as defined in Sec. 6.3.3.

T_R	Period, in seconds, of the residual mode of vibration of the *structure* in the direction under consideration, Sec. 13A.4.3.7.
T_S	S_{DI}/S_{DS}.
T_4	Net tension in steel cable due to *dead load,* prestress, *live load*, and seismic load.
t	Specified *wall* thickness dimension or least lateral dimension of a column (in. or mm) in Chapter 11.
t_c	Thickness of masonry cover over reinforcing bars measured from the surface of the masonry to the surface of the reinforcing bars (in. or mm) in Chapter 11.
V	The total design shear at the base of the *structure* in the direction of interest, as determined using the procedure of Sec. 5.3, including Sec. 5.4.1 (kip or kN).
V	Shear on a masonry section due to un-factored loads (lb or N) in Chapter 11.
V_b	The total lateral seismic design force or shear on elements of the *isolation system* or elements below the *isolation system* as prescribed by Eq. 13.3.4.1.
V_m	Shear strength provided by masonry (lb or N) in Chapter 11.
V_m	Design value of the seismic *base shear* of the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 5.4.5 or Sec. 13A.5.3.2 (kip or kN).
V_{mm}	Minimum allowable value of *base shear* permitted for design of the *seismic-force-resisting system* of the *structure* in the direction of interest, Sec. 13A.2.4.1 (kip or kN).
V_n	Nominal shear strength (lb or N) in Chapter 11.
V_R	Design value of the seismic *base shear* of the residual mode of vibration of the *structure* in a given direction, as determined in Sec. 13A.4.3.6 (kip or kN).
V_s	The total lateral seismic design factor or shear on elements above the *isolation system* as prescribed by Eq. 13.3.4.2.
V_s	Shear strength provided by shear reinforcement (lb or N) in Chapters 6 and 11.
V_t	The design value of the seismic *base shear* as determined in Chapter 5 (kip or N).
V_u	Required shear strength (lb or N) due to factored loads in Chapters 6 and 11.

V_x	The seismic design shear in *Story x* as determined in Chapter 5 (kip or kN).
V_I	The portion of seismic *base shear, V,* contributed by the fundamental mode as determined in Chapter 5 (kip or kN).
V_I	The design value of the seismic base shear of the fundamental mode in a given direction as determined in Chapter 5 (kip or kN).
ΔV	The reduction in V as determined in Chapter 5 (kip or kN).
ΔV_I	The reduction of V_I as determined in Chapter 5 (kp or kN).
v_s	The average shear wave velocity for the soils beneath the foundation at large strain levels as determined in Chapter 5 (ft/s or m/s).
$\bar{v}_s$	Average shear wave velocity in top one 100 ft (30 m); see Chapter 4.
v_{so}	The average shear wave velocity for the soils beneath the foundation at small strain levels as determined in Chapter 5 (ft/s or m/s).
W	The total gravity load of the *structure* defined in Chapter 5 (kip or kN). For calculation of a seismically isolated building *structure,* the period, W, is the total seismic dead load weight of the *structure* above the isolation system (kip or kN).
$\overline{W}$	The effective *gravity load* of the structure as defined in Sec. 5.8.2 (kip or kN).
$\overline{W}_1$	Effective fundamental mode *gravity load* of *structure* including portions of the live load determined in accordance with Eq. 5.4.5-2 for $m = 1$ (kip or kN).
$\overline{W}_R$	Effective residual mode *gravity load* of the *structure* determined in accordance with Eq. 13A.4.3.7-3 (kip or kN).
W_D	The energy dissipated per cycle at the *story displacement* for the *design earthquake.*
$\overline{W}_m$	The effective gravity load of m^{th} mode of vibration of the *structure* determined in Chapter 5 (kip or kN).
W_p	*Component* operating weight (lb or N).
w	Width of wood *shear panel* or *diaphragm* in Chapter 9 (ft or mm).
w	Moisture content (in percent), ASTM D2216-92.
w	The dimension of a diaphragm or *shear wall* in the direction of application of force.

w_i, w_x	The portion of the total *gravity load*, W, located or assigned to Level I or x (kip or kN).
z	The level under consideration; $x = 1$ designates the first level above the *base.*
x	Elevation in *structure* of a *component* addressed by Chapter 6.
y	Elevation difference between points of attachment in Chapter 6.
y	The distance, in ft (mm), between the center of rigidity of the *isolation system* rigidity and the element of interest measured perpendicular to the direction of seismic loading under consideration.
α	The relative weight density of the *structure* and the soil as determined in Chapter 5.
α	Angle between diagonal reinforcement and longitudinal axis of the member (degree or rad).
α	Velocity power term relating *damping device* force to *damping* device velocity.
β	Ratio of shear demand to shear capacity for the *story* between Level x and x-1.
β	The fraction of critical damping for the coupled *structure*-foundation system determined in Chapter 5.
β_D	Effective damping of the *isolation system* at the *design displacement* as prescribed by Eq. 13.9.5.2-1.
β_{eff}	Effective damping of the *isolation system* as prescribed by Eq. 13.9.3-2.
β_{HD}	Component of effective damping of the *structure* in the direction of interest due to post-yield hysteric behavior of the *seismic-force-resisting system* and elements of the *damping system* at effective ductility demand μ_D, Sec. 13A.3.2.2.
β_{HM}	Component of effective damping of the *structure* in the direction of interest due to post-yield hysteric behavior of the *seismic-force-resisting system* and elements of the *damping system* at effective ductility demand, μ_M, Sec. 13A.3.2.2.
β_I	Component of effective damping of the *structure* due to the inherent dissipation of energy by elements of the *structure*, at or just below the effective yield displacement of the *seismic-force-resisting system*, Sec. 13A.3.2.1.
β_M	Effective damping of the *isolation system* at the maximum *displacement* as prescribed by Eq. 13.9.5.2-2.

β_{mD} Total effective damping of the m^{th} mode of vibration of the *structure* in the direction of interest at the *design displacement*, Sec. 13A.3.2.

β_{mM} Total effective damping of the m^{th} mode of vibration of the *structure* in the direction of interest at the *maximum displacement* , Sec. 13A.3.2.

β_o The foundation damping factor as specified in Chapter 5.

β_R Total effective damping in the residual mode of vibration of the *structure* in the direction of interest, calculated in accordance with Sec. 13A.3.2 (μ_D =1.0 and μ_M = 1.0).

β_{Vm} Component of effective damping of the m^{th} mode of vibration of the *structure* in the direction of interest due to viscous dissipation of energy by the *damping system*, at or just below the effective yield displacement of the *seismic-force-resisting system*, Sec. 13A.3.2.3.

γ Lightweight concrete factor

γ The average unit weight of soil (lb/ft^3 or kg/m^3).

Δ The design *story* drift as determined in Chapter 5 (in. or mm).

Δ The *displacement* of the dissipation device and device supports across the story.

Δ Suspended ceiling lateral deflection (calculated) in Chapter 6 (in. or mm).

Δ_a The allowable *story* drift as specified in Chapter 5 (in. or mm).

Δ_D Total *design earthquake* story drift of the *structure* in the direction of interest, Sec. 13A.4.4.4 (in. or mm).

Δ_{ID} *Design earthquake* story drift due to the fundamental mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.4.4 (in. or mm).

Δ_M Total *maximum earthquake* story drift of the *structure* in the direction of interest, Sec. 13A.4.4.6 (in. or mm).

Δ_m The design modal *story* drift determined in Chapter 5 (in. or mm).

Δ_{mD} *Design earthquake* story drift due to the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.4.4 (in. or mm).

Δ_p Relative *displacement* that the *component* must be designed to accommodate as defined in Chapter 6.

Δ_{RD} *Design earthquake* story drift due to the residual mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.4.4 (in. or mm).

Δ^- Maximum positive *displacement* of an *isolator unit* during each cycle of prototype testing.

Δ^-	Maximum negative *displacement* of an *isolator unit* during each cycle of prototype testing.
δ_{avg}	The average of the *displacements* at the extreme points of the *structure* at Level x (in. or mm).
δ_{cr}	Deflation based on cracked section properties (in. or mm) in Chapter 11.
δ_i	Elastic deflection of Level i of the *structure* due to applied lateral force, f_i, Sec. 13A.4.3.3 (in. or mm).
δ_{iID}	Fundamental mode *design earthquake* deflection of Level i at the center of rigidity of the *structure* in the direction under consideration, Sec. 13A.4.4.2 (in. or mm).
δ_{iD}	Total *design earthquake* deflection of Level i at the center of rigidity of the *structure* in the direction under consideration, Sec. 13A.4.4.2 (in. or mm).
δ_{iM}	Total *maximum earthquake* deflection of Level i at the center of rigidity of the *structure* in the direction under consideration, Sec. 13A.4.4.2 (in. or mm).
δ_{iRD}	Residual mode *design earthquake* deflection of Level i at the center of rigidity of the *structure* in the direction under consideration, Sec. 13A.4.4.2 (in. or mm).
δ_{im}	Deflection of Level i in the m^{th} mode of vibration at the center of rigidity of the *structure* in the direction under consideration, Sec. 13A.5 (in. or mm).
δ_{max}	The maximum *displacement* at Level x (in. or mm).
δ_x	The deflection of Level x at the center of the mass at and above Level x as determined in Chapter 5 (in. or mm).
δ_{xe}	The deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis as specified in Chapter 5 (in. or mm).
δ_{xem}	The modal deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis as specified in Chapter 5 (in. or mm).
δ_{xmv}, δ_{xm}	The modal deflection of Level x at the center of the mass at and above Level x as determined in Chapter 5 (in. or mm).
δ_x, δ_{xl}	The deflection of Level x at the center of the mass at and above Level x as determined in Chapter 5 (in. or mm).
ϵ_{mu}	Maximum useable compressive strain of masonry (in./in. or mm/mm) in Chapter 11.
μ	Effective ductility demand on the *seismic-force-resisting system* in the direction of interest.

μ_D	Effective ductility demand on the *seismic-force-resisting system* in the direction of interest due to the *design earthquake*, Sec. 13A.4.
μ_M	Effective ductility demand on the *seismic-force-resisting system* in the direction of interest due to the *maximum considered earthquake*, Sec. 13A.4.
μ_{max}	Maximum allowable effective ductility demand on the *seismic-force-resisting system* due to *design earthquake*, Sec. 13A.3.5.
θ	The stability coefficient for *P-delta effects* as determined in Chapter 5.
τ	The overturning moment reduction factor.
ρ	A reliability coefficient based on the extent of structural redundancy present in a *building* as defined in Chapter 5.
ρ	Ratio of the area of reinforcement to the net cross-sectional area of masonry in a plane perpendicular to the reinforcement in Chapter 11.
ρ_b	Reinforcement ratio producing balanced strain conditions in Chapter 11.
ρ_h	Ratio of the area of shear reinforcement to the cross sectional area of masonry in a plane perpendicular to the reinforcement in Chapter 11.
ρ_s	Spiral reinforcement ratio for precast prestressed piles in Chapter 7.
ρ_v	Ratio of vertical or horizontal reinforcement in *walls*.
ρ_x	A reliability coefficient based on the extent of structural redundancy present in the *seismic-force-resisting system* of a *building* in the *x* direction.
λ	Time effect factor.
ϕ	The capacity reduction factor.
ϕ	Strength reduction factor in Chapters 6 and 11.
ϕ	Resistance factor for steel in Chapter 8 and wood in Chapter 12.
ϕ_{il}	Displacement amplitude at Level *i* of the fundamental mode of vibration of the *structure* in the direction of interest, normalized to unity at the roof level, Sec. 13A.4.3.3.
ϕ_{im}	The displacement amplitude at the i^{th} level of the *structure* for the fixed base condition in the m^{th} mode of vibration in the direction of interest normalized to unity at the roof level as determined in Chapter 5.
ϕ_{iR}	Displacement amplitude at Level *i* of the residual mode of vibration of the *structure* in the direction of interest normalized to unity at the roof level, Sec. 13A.4.3.7.

Γ_1	Participation factor of fundamental mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.3 or Sec. 13A.5.3.3 ($m = 1$).
Γ_m	Participation factor on the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 13A.5.3.3.
Γ_R	Participation factor of the residual mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.3.7.
∇_{1D}	*Design earthquake* story velocity due to the fundamental mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.4.5 (in/sec or mm/sec).
∇_D	Total *design earthquake* story velocity of the *structure* in the direction of interest, Sec. 13A.4.4.5 (in/sec or mm/sec).
∇_M	Total *maximum earthquake* story velocity of the *structure* in the direction of interest, Sec. 13A.4.4.6 (in/sec or mm/sec).
∇_{mD}	*Design earthquake* story velocity due to the m^{th} mode of vibration of the *structure* in the direction of interest, Sec. 13A.4.4.5 (in/sec or mm/sec).
Ω_0	Overstrength factor as defined in Table 5.2.2.
Ω	Factor of safety in Chapter 8.
$\sum E_D$	Total energy dissipated, in kip-inches (kN-mm), in the *isolation system* during a full cycle of response at the design *displacement*, D_D.
$\sum E_M$	Total energy dissipated, in kip-inches (kN-mm), on the *isolation system* during a full cycle of response at the maximum *displacement* , D_M.
$\sum \lvert F_D^{+} \rvert_{max}$	Sum, for all *isolator units*, of the maximum absolute value of force, in kips (kN), at a positive *displacement* equal to D_D.
$\sum \lvert F_D^{+} \rvert_{min}$	Sum, for all *isolator units,* of the minimum absolute value of force, in kips (kN), at a positive *displacement* equal to D_D.
$\sum \lvert F_D^{-} \rvert_{max}$	Sum, for all *isolator units*, of the maximum absolute value of force, in kips (kN), at a negative *displacement* equal to D_D.
$\sum \lvert F_D^{-} \rvert_{min}$	Sum, for all *isolator units*, of the minimum absolute value force , in kips (kN), at a negative *displacement* equal to D_D.
$\sum \lvert F_M^{-} \rvert_{max}$	Sum, for all *isolator units,* of the maximum absolute value of force, in kips (kN), at a positive *displacement* equal to D_M.
$\sum \lvert F_M^{-} \rvert_{min}$	Sum, for all *isolator units*, of the minimum absolute value of force, in kips (kN), at a positive *displacement* equal to D_M.

$\sum |F_M^-|_{max}$ Sum, for all *isolator units*, of the minimum absolute value of force, in kips (kN), at a negative *displacement* equal to D_M.

$\sum |F_M^-|_{min}$ Sum, for all *isolator units*, of the minimum absolute value of force, in kips (kN), at a negative *displacement* equal to D_M.

Chapter 3 Commentary

QUALITY ASSURANCE

3.1 SCOPE: Quality assurance (control and verification) for *structures* included in *Seismic Design Categories* C, D, E and F, is necessary do to the complexity of the *seismic-force resistive systems* and is important because of the serious consequences of the failure of *structures*. The level of quality assurance varies with the degree of seismic risk.

Quality Assurance requirements involve many aspects of the total design of *structures* and construction process; from the selection of the design team and their suitability for the project, to the capabilities of the construction contractor(s) and subcontractors, whether selected by qualification or by low bid. When *structures* are to be located in areas with probability of having damaging earthquake ground motion, the risk of loss of life demands adequate quality assurance to assure life safety. Unfortunately, earthquake related failures in recent seismic events that are directly traceable to poor design or quality control during construction are innumerable, and these deficiencies must be eliminated. The earthquake requirements included in the *Provisions* rely heavily upon the concept of adequate quality controls and verifications to ensure sound construction. It is important that all parties involved in the design and construction process understand and support the quality assurance requirements recommended in the *Provisions*.

The technological complexity of the design of modern *structures* necessitates employment of a team of *registered design professionals*. Each member in responsible charge of design of each element or system of the *structure* shall have been qualified and licensed by the jurisdiction to practice in their technical fields of practice. *Structures* located at a site with a potential for having damaging earthquake ground motion, must be designed to withstand the resulting seismic forces and accommodate element displacements.

Every element of a *structure* is a part of a continuous load path transmitting seismic forces from and to the foundations, which must be adequately strengthened and appropriately anchored to resist the seismic forces and accommodate the resulting displacements. Many of the failures in recent earthquakes have been attributed to weak links in the seismic force resisting load paths. Since the interconnection between adjacent elements of the *structure* often involves different *registered design professionals* and different construction trades during installation, it is imperative that these interconnections be adequately described in the *construction documents* and observed during installation. In order to accommodate these constraints and produce a coordinated design the *registered design professionals* must function as an integrated and well coordinated team.

The selection of the size and configuration of the *structure*, and the type of structural *seismic force resisting system(s)* selected (how rigid or ductile), can make a significant impact on the performance of the *structure* in an earthquake. Since the selection can affect the design and cost of construction of almost every element of the *structures*, it is essential that the entire design team be knowledgeable of and participate in these preliminary design decisions and appropriately accommodate them in their design. While not required by the *Provisions*, it is recommended that a *quality assurance plan* be prepared for the design process.

For quality assurance during construction, the following is included in the *Provisions*: (1) the *registered design professional(s)* in responsible charge of the design specifies the quality assurance requirements; (2) the prime contractor(s) exercises the control necessary to achieve the required quality: and (3) the *owner* is responsible for monitoring the construction process through *special inspections*, observations, and testing. It is important that each and every party involved recognizes their responsibilities, understands the procedures, and is capable of carrying them out. Because the contractor and specialty subcontractors are performing the work and exercising control of quality, it is essential that the *special inspections* and tests be performed by someone not in their direct employ. For this reason, the *special inspectors* are the *owner's* inspectors, and serve at the discretion of the authority having jurisdiction. When the *owner* is also the contractor, the *owner*, to avoid a potential conflict of interest, must engage independent agencies to conduct the *special inspections* and tests rather than try to qualify his own employees for that purpose.

The contractual responsibilities during the construction phase, vary from project to project, depending on the *structure*, and the desires of the *owner*. The majority of *building owners* use the standard contract forms published by the American Institute of Architects (AIA) or the Engineers' Joint Contract Documents Committee (EJCDC), or one modeled therefrom, which includes specific construction phase responsibilities.

The *registered design professional* in responsible charge for each portion of the project is the most knowledgeable, and frequently the only person available for assuring appropriate conformance with the intent of the design as conveyed in the *construction documents*. It is essential that a *registered design professional* be sufficiently involved during the construction phase of the project to assure general conformance with the approved *construction documents*. Courts are ruling more frequently that the above responsibilities remain that of the *registered design professional* in responsible charge of the design, regardless of the language included in the contract for professional services.

The quality assurance requirements included in Chapter 3 of the *Provisions* are the minimum requirements. It could be the decision of the *owner* or *registered design professional* to include more stringent quality assurance requirements. The primary method for achieving quality assurance is through the use of *special inspectors* and testing agencies.

Registered design professional(s) in responsible charge, or their employees, may perform the *special inspections*, when approved by the authority having jurisdiction. Increased involvement by the *registered design professional* in responsible charge allows for early detection of problems during construction when they can more easily be resolved.

3.2 QUALITY ASSURANCE: Because of the complexity of design and construction for *structures* included in *Seismic Design Categories* C, D, E and F, it is necessary to provide a comprehensive written *quality assurance plan* to assure adequate quality controls and verification during construction. Each portion of the *quality assurance plan* is required to be prepared by the *registered design professional* responsible for the design of the *seismic-force-resisting system(s)* and other *designated seismic system(s)* that are subject to requirements for quality assurance. When completed, the *quality assurance plan* must be submitted to the *owner*, and to the authority having jurisdiction.

The performance for quality control of the contractors and subcontractors varies project to project. The *quality assurance plan* is an opportunity for the *registered design professional* to delineate the types and frequency of testing and inspections, and the extent of the *structural observations* to be performed during the construction process, to assure that the construction is in conformance with the approved *construction documents*. Special attention should be given in the *quality assurance plan* for projects with higher *occupancy importance factors*.

The authority having jurisdiction shall approve the *quality assurance plan* and shall obtain from each contractor a written statement that the contractor understands the requirements of the *quality assurance plan* and will exercise the necessary control to obtain conformance. The exact methods of control are the responsibility of the individual contractors, subject to approval by the authority having jurisdiction. *Special inspections,* in addition to those included in the *quality assurance plan*, may required by the authority having jurisdiction to provided assurance that there is compliance with the approved *construction documents*.

A *quality assurance plan* is not required for some low-rise multi-family dwellings, commercial, mercantile, and office buildings that are included in *Seismic Use Group* I, as indicated in the exception to Sec. 3.2. The exception is also limited to those *structures* that do not have any of the delineated irregularities. Any *structure* that does not satisfy all of the criteria included in the exception or is not otherwise exempted by the *Provisions* is required to have a *quality assurance plan*. It is important to emphasize that this exemption only applies to the preparation of a *quality assurance plan*. All *special inspections* and testing that are otherwise required by the *Provisions* are not exempt and must be performed.

3.3 SPECIAL INSPECTION: *Special inspection* is the monitoring of materials and workmanship that are critical to the integrity of the *structure*. The requirements listed in this section, from foundation systems through cold formed steel framing, have been included in the national model codes of many years. It is a premise of the *Provisions* that there will be an adequate supply of knowledgeable and experienced inspectors available to provide the necessary *special inspections* for the structural categories of work. Special training programs may have to be developed and implemented for the nonstructural categories.

A *special inspector* is a person approved by the authority having jurisdiction as being qualified to perform *special inspections* for the category of work involved. As a guide to the authority having jurisdiction, it is contemplated that the *special inspector* is to be one of the following:

1. A person employed and supervised by the *registered design professional* in responsible charge for the design of the *designated seismic system* or the *seismic-force-resisting system* for which the *special inspector* is engaged.

2. A person employed by an approved inspection and/or testing agency who is under the direct supervision of a *registered design professional* also employed by the same agency, using inspectors or technicians qualified by recognized industry organizations as approved by the authority having jurisdiction.

3. A manufacturer or fabricator of *components*, equipment, or machinery that has been approved for manufacturing *components* that satisfy seismic safety standards and that maintain a *quality assurance plan* approved by authority having jurisdiction. The

manufacturer or fabricator is required to provide evidence of such approval by clearly marking on each *designated seismic system* or *seismic-force-resisting system component* shipped to the construction site.

The extent and duration of *special inspections*, types of testing, and the frequency of the testing must be clearly delineated in the *quality assurance plan*. In some instances the *Provisions* allow *periodic special inspection* versus *continuous special inspection*. When *periodic special inspections* are allowed, the *Provisions* do not state specific requirements for frequency of periodic inspection, but give minimum stages of construction at which inspection is required for a particular category of work. The *quality assurance plan* should generally indicate the timing and extent of any *periodic special inspections* required by the *Provisions*.

3.3.9 Architectural Components: It is anticipated that the minimum requirements for architectural *components* (e.g. exterior cladding) are satisfied when that the method of anchoring *components* and the number, spacing, and types of fasteners actually used conforms with approved *construction documents*. It is noted that such *special inspection* requirements are only for those *components* in *Seismic Design Categories* D, or E, or F.

3.3.10 Mechanical and Electrical Components: It is anticipated that the minimum requirements for mechanical and electrical *components* are satisfied when the method of anchoring *components* and the number, spacing, and types of fasteners actually used conforms with the approved *construction documents*. It is noted that such *special inspection* requirements are for selected electrical, lighting, piping and ductwork *components* in all *Seismic Design Categories* except A and B, and for all electrical equipment in *Seismic Design Categories* E and F.

3.4 TESTING: Compliance with nationally recognized test standards provides the authority having jurisdiction and the *owner* a means to determine the acceptability of materials and their placement. Most test standards for materials are developed and maintained by the American Society of Testing and Materials (ASTM). Through their reference in model building codes and material specifications, ASTM Standards and other standard testing procedures provide a universal measure for acceptance of materials and construction. The *Provisions* and the model building codes require that standard tests be performed by an approved testing agency.

Special inspector(s) are responsible for the observation and verification of the testing procedures performed in the field. Special inspectors determine compliance with test standards based on their interpretation of the standards, as measured against acceptance criteria that are included in the *construction documents* and the *quality assurance* plan.

Test standards also prescribe responsibilities for others. For example, ASTM A 706 specification for low-alloy steel reinforcing bars requires the manufacturer to report the chemical composition and carbon equivalent of the material. In addition, the ANSI/AWS D1.4 Welding Code requires the contractor to prepare written specifications for the welding of reinforcing bars. It is necessary, therefore, that each member of the construction team has a thorough knowledge of the specified test standards that cover their particular work.

3.4.5 Mechanical and Electrical Equipment: The registered design professional should consider requirements to demonstrate the seismic performance of mechanical and electrical *components* critical to the post–earthquake life safety of the occupants. Any requirements should be clearly indicated on the construction documents. Any currently accepted technology should be acceptable to demonstrate compliance with the requirements.

3.5 STRUCTURAL OBSERVATIONS: The purpose of *structural observations* is to allow the *registered design professional(s)* in responsible charge or other *registered design professional(s)* to visit the site to observe the *seismic-force-resisting systems*. Observations include verifying the *seismic-force-resisting system* is constructed in general conformance with the *construction documents*, and the intent of the design has been accomplished and that a complete lateral load path exists.

Every effort shall be made to have the *registered design professional* in responsible charge make the observations. If another *registered design professional* performs the observations he is expected to be familiar with the *construction documents* and the design concept.

3.6 REPORTING AND COMPLIANCE PROCEDURES: The purpose of this section is to keep parties as delineated in the *Provisions* informed of the *special inspector's* observations and the contractor's corrections.

Chapter 4 Commentary

GROUND MOTION

4.1 PROCEDURES FOR DETERMINING MAXIMUM CONSIDERED EARTHQUAKE AND DESIGN EARTHQUAKE GROUND MOTION ACCELERATIONS AND RESPONSE SPECTRA: This section sets alternative procedures for determining ground shaking parameters for use in the design process. The design requirements generally use response spectra to represent ground motions in the design process. For the purposes of the *Provisions*, these spectra are permitted to be determined using either a generalized procedure in which mapped seismic response acceleration parameters are referred to or by site-specific procedures. The generalized procedure in which mapped values are used is described in Sec. 4.1.2. The site-specific procedure is described in Sec. 4.1.3.

4.1.1 Maximum Considered Earthquake Ground Motions: The *Provisions* are intended to provide uniform levels of performance for structures, depending on their occupancy and use and the risk to society inherent in their failure. Sec. 1.3 of the *Provisions* establishes a series of Seismic Use Groups that are used to categorize structures based on the specific Seismic Design Category. It is the intent of the *Provisions* that a uniform margin of failure to meet the seismic design criteria be provided for all structures within a given Seismic Use Group.

In past editions of the *Provisions*, seismic hazards around the nation were defined at a uniform 10 percent probability of exceedance in 50 years and the design requirements were based on assigning a structure to a Seismic Hazard Exposure Group and a Seismic Performance Category. While this approach provided for a uniform likelihood throughout the nation that the design ground motion would not be exceeded, it did not provide for a uniform margin of failure for structures designed for that ground motion. The reason for this is that the rate of change of earthquake ground motion versus likelihood is not constant in different regions of the United States.

The approach adopted in the *Provisions* is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this, ground motion hazards are defined in terms of maximum considered earthquake ground motions. The maximum considered earthquake ground motions are based on a set of rules that depend on the seismicity of an individual region. The design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the *Provisions*. This lower bound was judged, based on experience, to be about a factor of 1.5 in ground motion. Consequently, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (2/3) of the maximum considered earthquake ground motion.

For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (return period of about 2500 years). While stronger shaking than this could occur, it was judged that it would be economically impractical to design for such very rare ground motions and the selection of the 2 percent in 50 years likelihood as the maximum considered earthquake ground motion would result in acceptable levels of seismic safety for the nation.

In regions of high seismicity, such as coastal California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well defined fault systems. Ground shaking calculated at a 2 percent in 50 years likelihood would be much larger than that which would be expected based on the characteristic magnitudes of earthquakes on these known active faults. This is because these major active faults can produce characteristic earthquakes every few hundred years. For these regions, it is considered more appropriate to directly determine maximum considered earthquake ground motions based on the characteristic earthquakes of these defined faults. In order to provide for an appropriate level of conservatism in the design process, when this approach to calculation of the maximum considered earthquake ground motion is used, the median estimate of ground motion resulting for the characteristic event is multiplied by 1.5.

Sec. 4.1.1 of the *Provisions* defines the maximum considered earthquake ground motion in terms of the mapped values of the spectral response acceleration at short periods, S_S, and at 1 second, S_1, for Site Class B sites. These values may be obtained directly from Maps 1 through 24, respectively. A detailed explanation for the development of Maps 1 through 24 appears as Appendix A of this *Commentary* volume. The logic by which these maps were created, as described above and in Appendix A, is also included in the *Provisions* under Sec 4.1.3, Site-Specific Procedures, so that registered design professionals performing such a study may use methods consistent with those that served as the basis for developing the maps.

4.1.2 General Procedure for Determining Maximum Considered Earthquake Ground Motions and Design Spectral Response Accelerations: This section provides the procedure for obtaining design site spectral response accelerations using the maps provided with the *Provisions*. Most buildings and structures will be designed using the equivalent lateral force technique of Sec. 5.4, and this general procedure to determine the design spectral response acceleration parameters, S_{DS} and S_{D1}, that are directly used in that procedure. Some structures will be designed using the modal analysis procedures of Sec. 5.5. This section also provides for the development of a general response spectrum, which may be used directly in the modal analysis procedure, from the design spectral response acceleration parameters, S_{DS} and S_{D1}.

Maps 1 and 2 respectively provide two parameters S_S and S_1 based on a national seismic hazard study conducted by the U.S. Geological Survey. For most buildings and sites, they provide a suitably accurate estimate of the maximum considered earthquake ground shaking for design purposes. For some sites, with special soil conditions or for some buildings with special design requirements, it may be more appropriate to determine a site specific estimate of the maximum considered earthquake ground shaking response accelerations. Sec. 4.1.3 provides guidance on site-specific procedures.

S_S is the mapped value, from Map 1 of the 5 percent damped maximum considered earthquake spectral response acceleration, for short period structures founded on Class B, firm rock, sites. The short period acceleration has been determined at a period 0.2 seconds. This is because it was concluded that 0.2 seconds was reasonably representative of the shortest effective period of buildings and structures that are designed by the *Provisions*, considering the effects of soil compliance, foundation rocking and other factors typically neglected in structural analysis.

Similarly, S_I is the mapped value from Map 2 of the 5 percent damped maximum considered earthquake spectral response acceleration at a period of 1 second on Site Class B. The spectral response acceleration at periods other than 1 second can typically be derived from the acceleration at 1 second. Consequently, these two response acceleration parameters, S_S and $S_{I,}$ are sufficient to define an entire response spectrum for the period range of importance for most buildings and structures, for maximum considered earthquake ground shaking on Class B sites.

In order to obtain acceleration response parameters that are appropriate for sites with other characteristics, it is necessary to modify the S_S and S_I values, as indicated in Sec.4.1.2.4. This modification is performed with the use of two coefficients, F_a and F_v which respectively scale the S_S and S_I values determined for firm rock sites to appropriate values for other site conditions. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects are designated respectively, S_{MS} and S_{MI}, for short period and 1 second period response. As described above, structural design in the *Provisions* is performed for earthquake demands that are 2/3 of the maximum considered earthquake response spectra. Two additional parameters, S_{DS} and S_{DI} are used to define the acceleration response spectrum for this design level event. These are taken, respectively as 2/3 of the maximum considered earthquake values S_{MS} and S_{MI}, and completely define a design response spectrum for sites of any characteristics.

Sec. 4.1.2.1 provides a categorization of the various classes of site conditions, as they affect the design response acceleration parameters. Sec. 4.1.2.2 describes the method by which sites can be classified according as belonging to one of these Site Classes. Sec. 4.1.2.3 provides definitions of some site parameters referenced in the preceding section.

4.1.2.1 Site Class Definitions: It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design, and most countries considering these effects have developed different design criteria for several different soil conditions. The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in developing the 1994 *Provisions*. Evidence of the effects of local soil conditions has been observed globally including eastern North America. An example of the latter is a pocket of high intensity reported on soft soils in Shawinigan, Quebec, approximately 155 miles (250 km) from the 1925 Charlevoix magnitude 7 earthquake (Milne and Davenport, 1969).

The Applied Technology Council (ATC) study that generated the preliminary version of the *Provisions* provided for the use of three Soil Profile Types considered, in the late 1970s, to be different enough in seismic response to warrant separate site coefficients (S factors) and experience from the September 1985 Mexico City earthquake prompted the addition of a fourth Soil Profile Type. These have been revised for the 1994 *Provisions* to conform to the experiences of the Mexico City and the 1989 Loma Prieta earthquake in California as well as to other observations and studies showing the effects of level of shaking, rock stiffness, and soil type, stiffness and depth on the amplification of ground motions at short and long periods. The resulting use of higher seismic coefficients in areas of lower shaking and the addition of a "hard rock" category in the 1994 *Provisions* better reflect the conditions in some parts of the country and incorporate recent efforts toward a seismic code for New York City (Jacob, 1990 and 1991). The need for improvement in codifying site effects was discussed at a 1991 National Center for Earthquake Engineering Research (NCEER) workshop devoted to the subject (Whitman, 1992), which made several general recommendations. At the urging of Robert V. Whitman, a committee was formed during that workshop to pursue resolution of pending issues and develop specific code recommendations. Serving on this committee were M. S. Power (chairman), R. D. Borcherdt, C. B. Crouse, R. Dobry, I. M. Idriss, W. B. Joyner, G. R. Martin, E. E. Rinne, and R. B. Seed. The committee collected information, guided related research, discussed the issues, and organized a November 1992 Site Response Workshop in Los Angeles (Martin, 1994). This workshop discussed the results of a number of empirical and analytical studies and approved consensus recommendations that form the basis for the 1994 *Provisions*.

Amplification of Peak Ground Acceleration: Seed and coworkers (1976a) conducted a statistical study of peak accelerations developed at locations with different site conditions using 147 records from each western U.S. earthquake of about magnitude 6.5. Based on these results, judgment and analysis, they proposed the acceleration relations of Figure C4.1.2-1a that are applicable to any earthquake magnitude of engineering interest. It must be noted that the data base of that study did not include any soft clay sites and, thus, the corresponding curve in the figure was based on the authors' experience and, consequently, was somewhat more speculative.

Idriss (1990a and 1990b), using data from the 1985 Mexico City and 1989 Loma Prieta earthquakes, recently modified the curve for soft soil sites as shown in Figure C4.1.2-1b. In these earthquakes, low maximum rock accelerations of 0.05g to 0.10g were amplified by factors of from about 1.5 to 4 at sites containing soft clay layers ranging in thickness from a few feet to more than a hundred feet and having depths of rock up to several hundred feet. As shown by the data and site response calculations included in Figure C4.1.2-1b, the average amplification factor for soft soil sites tends to decrease as the rock acceleration increases--from 2.5 to 3 at low accelerations to about 1.0 for a rock acceleration of 0.4g. Since this effect is directly related to the nonlinear stress-strain behavior in the soil as the acceleration increases, the curve in Figure C4.1.2-1b can be applied in first approximation to any earthquake magnitude of engineering interest.

It is clear from Figure C4.1.2-1b that low peak accelerations can be amplified several times at soil sites, especially those containing soft layers and where the rock is not very deep. On the other hand, larger peak accelerations can be amplified to a lesser degree and can even be slightly deamplified at very high rock accelerations. In addition to peak rock acceleration, a number of factors including soil softness and layering play a role in the degree of amplification. One important factor is the impedance contrast between soil and underlying rock.

Spectral Shapes: Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of the spectral shapes developed on such soils close to the seismic source zone in past earthquakes (Seed et al., 1976a and 1976b; Hayashi et al., 1971).

The mean spectral shapes determined directly from the study by Seed and coworkers (1976b), based on 104 records from 21 earthquakes in the western part of the United States, Japan and Turkey, are shown in Figure C4.1.2-2. The ranges of magnitudes and peak accelerations covered by this data base are 5.0 to 7.8 and 0.04g to 0.43g, respectively. All spectra used to generate the mean curve for soft to medium clay and sand in Figure C4.1.2-2 correspond to rather low peak accelerations in the soil (less than 0.10g). The spectral shapes in the figure also were compared with the studies of spectral shapes conducted by Newmark et al. (1973), Blume et al. (1973), and Mohraz (1976) and with studies for use in model building regulations. It was considered appropriate to simplify the form of the curves to a family of three by combining the spectra for rock and stiff soil conditions leading to the normalized spectral curves shown in Figure C4.1.2-3. The curves in this figure therefore apply to the three soil conditions in the original version (1985) of the *Provisions*.

The three conditions corresponding to the three lines in Figure C4.1.3-3 plus a fourth condition introduced following the 1985 Mexico City earthquake are described as follows:

1. Soil Profile Type S_1--A soil profile with either: (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 ft/s (762 m/s) or (2) stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.
2. Soil Profile Type S_2--A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.
3. Soil Profile Type S_3--A soil profile containing 20 to 40 ft (6 to 12 m) in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils.
4. Soil Profile Type S_4--A soil profile characterized by a shear wave velocity of less than 500 ft/sec (152 m/s) containing more than 40 ft (12 m) of soft clays or silts.

The post-Loma Prieta studies (Martin, 1994) have resulted in considerable modification of these profile types resulting in the Soil Profile Types in the 1994 *Provisions*, A through F.

Response of Soft Sites to Low Rock Accelerations: Earthquake records on soft to medium clay sites subjected to low acceleration levels indicate that the soil/rock amplification factors for long-period spectral accelerations can be significantly larger than those in Figures C4.1.2-1 and C4.1.2-2 (Seed et al., 1974). Furthermore, the largest amplification often occurs at the natural period of the soil deposit. In Mexico City in 1985, the maximum rock acceleration was amplified four times by a soft clay deposit that would have been classified as S_4 whereas the spectral amplitudes were about 15 to 20 times larger than on rock at a period near 2 sec. In other parts of the valley where the clay is thicker, the spectral amplitudes at periods ranging between 3 and 4 sec also were amplified about 15 times, but the damage was less due to the low rock motion intensity at these very long periods (Seed et al., 1988). Inspection of the records obtained at some soft clay sites during the 1989 Loma Prieta earthquake indicates a maximum amplification of long-period spectral amplitudes of the order of three to six times.

Figure C4.1.2-4 shows a comparison of average response spectra measured on rock and soft soil sites in San Francisco and Oakland during this magnitude 7.1 earthquake. A preliminary study of the Loma Prieta records at one 285-ft (87 m) soil deposit on rock containing a 55-ft (17 m) soft to medium stiff clay layer (Treasure Island) seems to suggest that the largest soil/rock amplification of response spectra occurred at the natural period of the soil deposit, similarly to Mexico City (Seed et al., 1990).

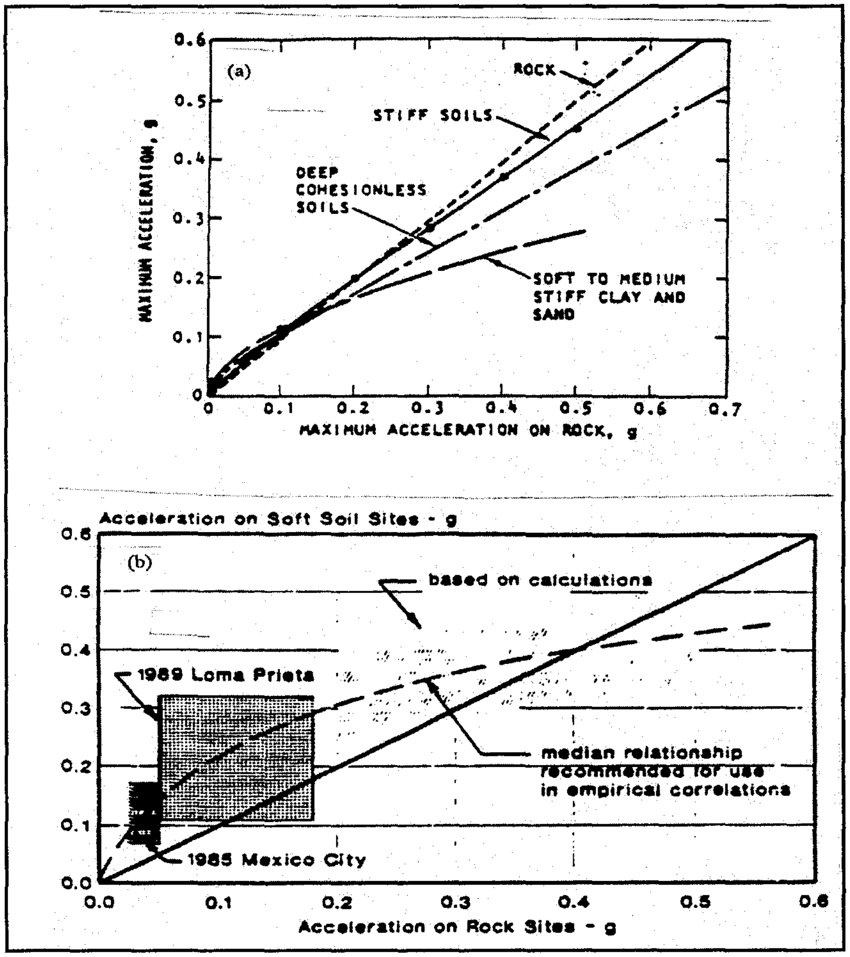


FIGURE C4.1.2-1 relationships between maximum acceleration on rock and other local site conditions: (top) Seed et al., 1976a, and (bottom) Idriss, 1990a and 199b.

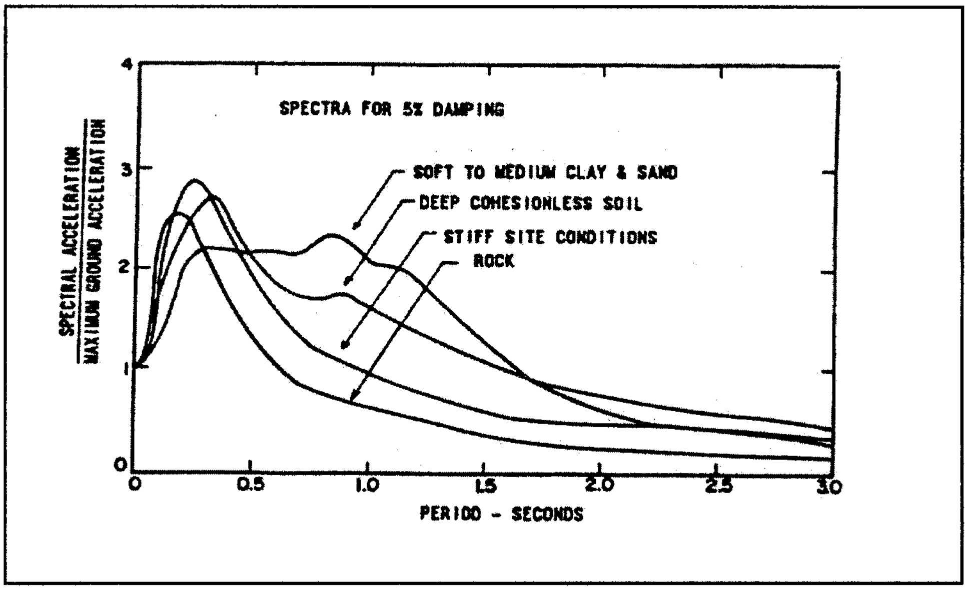


FIGURE C4.1.2-2 Average acceleration spectra for different site conditions (Seed et al., 1976a and 1976b).

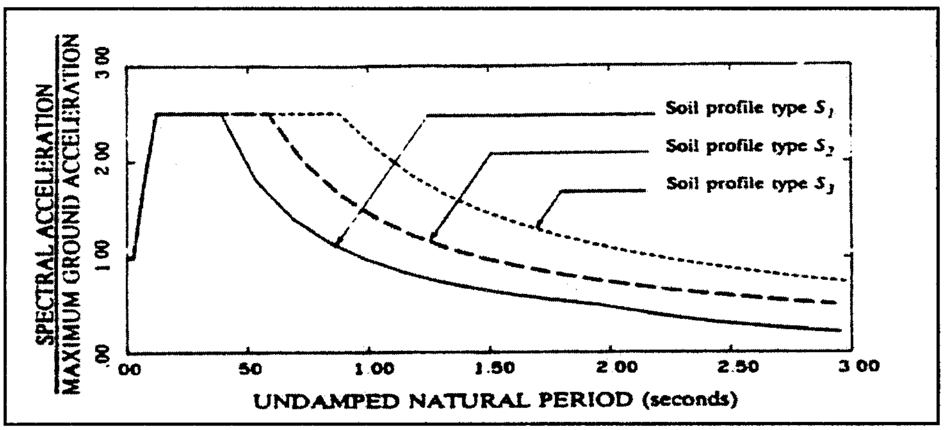


FIGURE C4.1.2-3 Normalized response spectra, damping = 0.05.

Some relevant theoretical and experimental findings are reviewed briefly below to clarify the role of key site parameters in determining the magnitude of the soil/rock amplification of spectral ordinates at long periods for sites containing soft layers. These parameters are the thickness of the soft soil, the shear wave velocity of the soft soil, the soil/rock impedance ratio (*IR*), the layering and properties of the stiffer soil between soft layer and rock, and the modulus and damping properties of the soft soil. The basic assumptions used are those typically used in one-dimensional site response analyses and, thus, the conclusions drawn are restricted to sites where these conditions are fulfilled (i.e., flat sites with horizontal layering of significant extension and far from rock outcrops and with a clear soil-rock interface at a depth not exceeding several hundred feet).

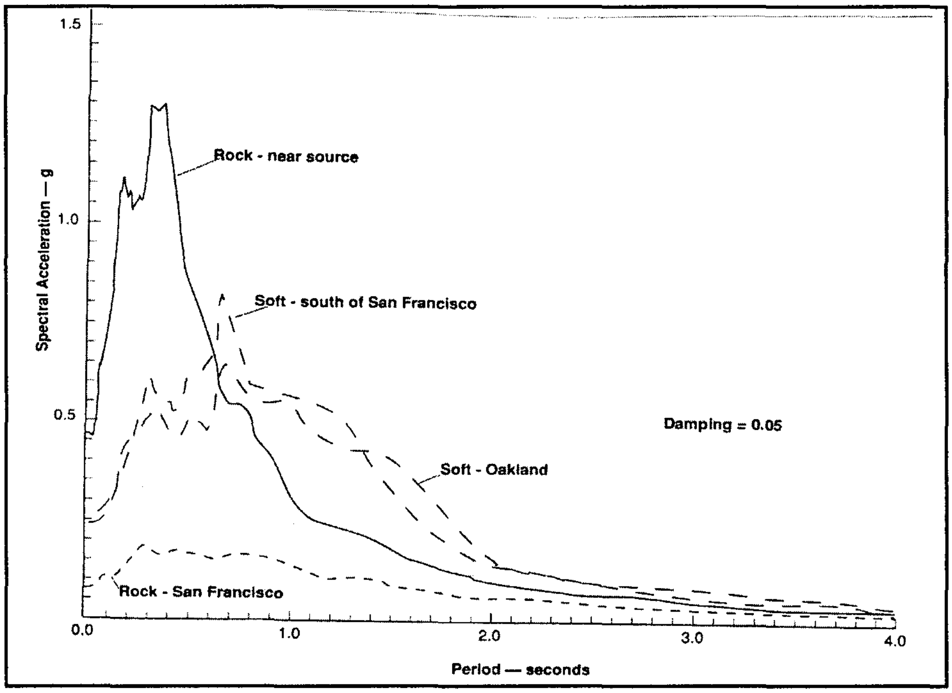


FIGURE C4.1.2-4 Average spectra recorded during 1989 Loma Prieta earthquake at rock sites and soft soil sites (Housner, 1990).

The uniform layer on elastic rock sketched in Figure C4.1.2-5 is subjected to a vertically propagating shear wave representing the earthquake. The soil layer is assumed to behave linearly and it has a thickness h, total (saturated) unit weight g_s, shear wave velocity v_s, and internal damping ratio b_s. The rock has total unit weight g_r, shear wave velocity v_r, and zero damping.

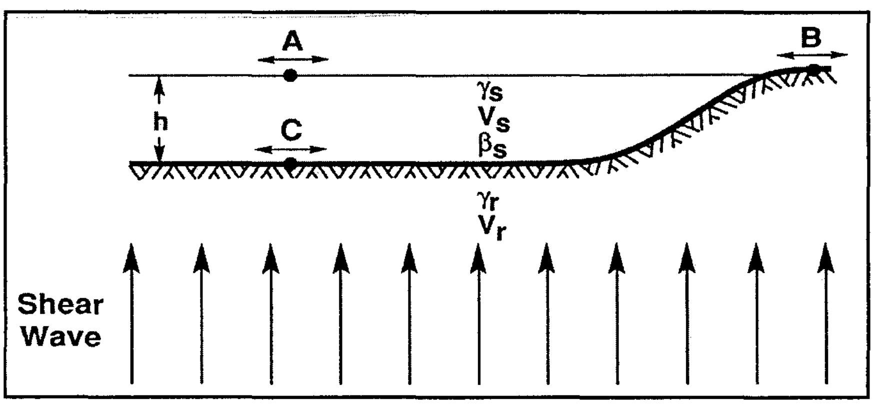


FIGURE C4.1.2.5 Uniform soil layer on elastic rock subjected to vertical shear waves.

Due to the soil-rock interaction effect, the motion at the soil-rock interface C is different (typically less) from that at the rock outcrop B. Only if the rock is rigid (v_s = ¥) are the motions at C and B equal. Of interest here is the ratio between the motions on top of the soil (point A) and on the rock outcrop (point B).

When the acceleration at B is a harmonic motion of frequency f (cps) and amplitude a_B, the acceleration at A is also harmonic of the same frequency and amplitude a_A. The amplification ratio a_A/a_B is a function of the ratio of frequencies $f/(v_s/4h)$, of the soil damping b_s, and of the rock/soil impedance ratio which is equal to $g_r v_r / g_s v_s$. Figure C4.1.2-6 presents a_A/a_B calculated for a layer with h = 100 ft (30.5 m), $v_s/4h$ = 1.88 cps, and IR = 6.7 (Roesset, 1977). The maximum amplification occurs essentially at the natural frequency of the layer, $f_{soil} = V_s/4h$, and is approximately equal to:

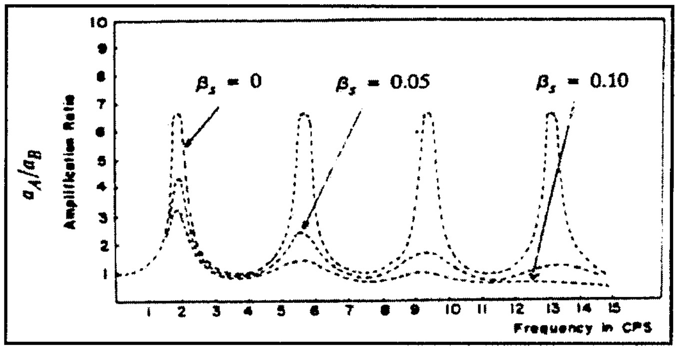


FIGURE C4.1.2-6 Amplification ratio soil/rock for h = 100 ft (30.5m), V_s =1.88 cps, and IR =6.7 (Roesset, 1977).

$$\left(\frac{a_A}{a_B}\right)_{\max} = \frac{I}{\left(\frac{1}{IR}\right) + \left(\frac{\pi}{2}\right)\beta_s} \qquad \text{(C4.1.2-1)}$$

That is, the maximum soil/rock amplification for steady-state harmonic motion in this simple model depends on two factors--b_s and IR. When IR = ¥ (rigid rock), the only way the system can dissipate energy is in the soil and $(a_A/a_B)_{max} = 2/pb_s$ can be very large. For example, if IR = ¥ and b_s = 0.04, $(a_A/a_B)_{max}$ = 16. If IR decreases, the amplification $(a_A/a_B)_{max}$ also decreases. For example, if IR = 15 and b_s = 0.04, the amplification is cut in half, $(a_A/a_B)_{max}$ = 8.

Another way of expressing the contribution of the impedance ratio IR in Eq. C4.1.2-1 is as an "additional equivalent soil damping" with a total damping b_{tot} in the system at its natural frequency:

$$\beta_{tot} \approx \beta_s + \left(\frac{2}{\pi IR}\right) \qquad \text{(C4.1.2-2)}$$

Eq. C4.1.2-2 is very important since the maximum amplification $(a_A/a_B)_{max}$ is always inversely proportional to b_{tot}, not only for the case of the uniform layer but also for other soil profiles on rock. b_{tot} always includes an internal damping contribution (b_s) and a second term reflecting the rock-soil impedance contrast IR although the specific definition of IR and the numerical factor $2/p$ generally will change depending on the profile. When a soft layer lies on top of a significant thickness of stiffer soil followed by rock, Eq. C4.1.2-2 is still qualitatively valid, but the calculations are more complicated. In that case, the impedance contrast must consider the whole soil profile and, thus, both soft and stiff soils play a role in determining b_{tot} and $(a_A/a_B)_{max}$. Also, the maximum amplification may occur at the natural frequency of the soft layer, of the whole profile, or at some other frequency.

Two-Factor Approach and the 1992 Site Response Workshop: The recommendations developed during the NCEER/SEAOC/BSSC Site Response Workshop mentioned above were summarized by Rinne and Dobry (1992) and are reprinted as Appendix F of this commentary to provide the reader with a better understanding of the thinking behind the current *Provisions*. Some additional background information taken mostly from the proceedings of that workshop (Martin, 1994) is included below.

As discussed above, soil sites generally amplify more the rock spectral accelerations at long periods than at short periods and, for a severe level of shaking (S_S >> 1.0g; S_1 >> 0.4g), the short-period amplification or deamplification is small; this was the basis for the use in the previous versions of the *Provisions*. However, the evidence that short-period accelerations including the peak acceleration can be amplified several times, especially at soft sites subjected to low levels of shaking, suggested the replacement of the normalized spectrum approach by the two-factor approach sketched in Figure C4.1.2-7. In this approach, adopted in the 1994 *Provisions*, the short-period plateau, represented by S_{MS}, is multiplied by a short-period site coefficient F_a and the long period curve represented by S_{M1}/T is multiplied by a long-period site coefficient F_v. Both F_a and F_v depend on the site conditions and on the level of shaking, defined respectively by the values of S_S and S_1.

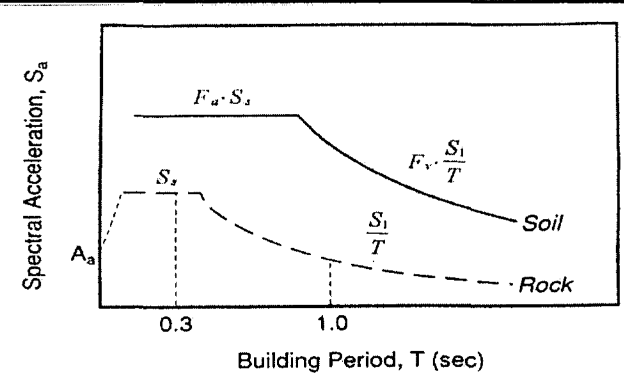


FIGURE C4.1.2.7 Two-factor approach to local site response.

Strong-motion recordings, as obtained from the Loma Prieta earthquake of October 17, 1989, provide important quantitative measures of the *in situ* response of a variety of geologic deposits to damaging levels of shaking. Average amplification factors derived from these data with respect to "firm to hard rock" for short-period (0.1-0.5 sec), intermediate-period (0.5-1.5 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands show that a short- and mid-period factor are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach summarized in Figure C4.1.2-7. Empirical regression curves fit to these amplification data as a function of mean shear wave velocity at the site are shown in Figure C4.1.2-8.

These curves provide empirical estimates of the site coefficients F_a and F_v as a function of mean shear wave velocity for input ground motion levels near 0.1g (Borcherdt and Glassmoyer, 1993). The empirical amplification factors predicted by these curves are in good agreement with those derived independently based on numerical modeling of the Loma Prieta strong-motion data (Seed et al., 1992) and those derived from parametric studies of several hundred soil profiles (Dobry et al., 1994b). These empirical relations are consistent with theory in that they imply that the average amplification at a site increases as the rock/soil impedance ratio (IR) increases, similar to the trend described by Eq. C4.1.2-1. They also are consistent with observed correlations between amplification and shear velocity for soft clays in Mexico City (Ordaz and Arciniegas, 1992). These short- and mid-period amplification factors implied by the Loma Prieta strong-motion data and related calculations for the same earthquake by Joyner et al. (1994) as well as modeling results at the 0.1g level provided the basis for the consensus values provided in Tables 4.1.2a and 4.1.2b. Values at higher levels were initially determined from modeling results for soft clays derived by Seed (1994) with values for intermediate soil conditions derived by linear extrapolation. A rigorous framework for extrapolation of the Loma Prieta results consistent with the results in Tables C4.1.2a and C4.1.2b is given in the following paragraph.

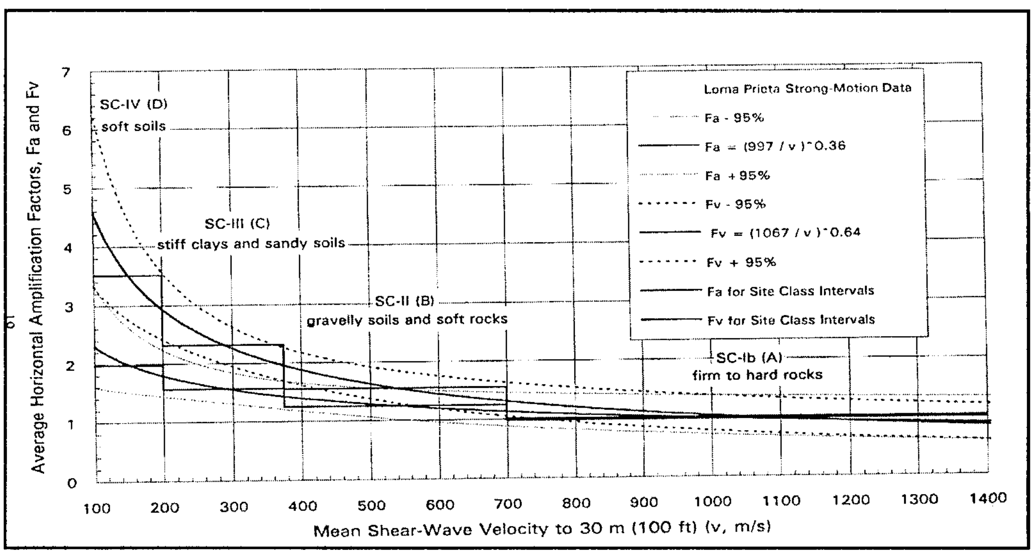


FIGURE C4.1.2-8 Short period F_a and mid-period F_v amplification factors with respect to "firm to hard" rock plotted as a continuous function of mean shear wave velocity using the regression equations derived from the strong-motion recordings of the Loma Prieta earthquake. The 95 percent confidence intervals for the ordinate to the true population regression line and the amplification factors for the simplified site classes also are shown (Borcherdt, 1994).

Extrapolation of amplification estimates at the 0.1g level as derived from the Loma Prieta earthquake must necessarily be based on laboratory and theoretical modeling considerations because few or no strong-motion recordings have been obtained at higher levels of motion, especially on soft soil deposits. Resulting estimates should be consistent with other relations between large rock and soil motions and local site conditions as summarized in Figure C4.1.2-1. The form of the regression curve in Figure C4.1.2-8 suggests a simple and well defined procedure for extrapolation. It shows that the functional relationship between the logarithms of amplification and mean shear velocity is a straight line (Borcherdt, 1993). Consequently, as the amplification factor for "firm to hard" rock is necessarily unity, the extrapolation problem is determined by specification of the amplification factors at successively higher levels of motion for the soft-soil site class. For input ground motion levels near 0.1g, Borcherdt (1993) began with amplification levels specified by the empirical regression curves (Figure C4.1.2-8) for the Loma Prieta strong-motion data. Higher levels of motion were inferred from laboratory and numerical modeling results (Seed et al., 1992; Dobry et al., 1994a). The resulting short-period (F_a) and mid-period (F_v) site coefficients as a function of mean shear velocity (v--labeled $_s$ elsewhere in this *Commentary* and in the *Provisions*) and input ground motion level (I_a) specified with respect to "firm to hard" rock are given in Figure C4.1.2-9 and plotted with logarithmic scales. These expressions state that the average amplification at a site is equal to the "rock-soil" impedance ratio raised to an exponent (*ma* or *mv*). These exponents are defined as the slope of the straight line determined by the logarithms of the amplification factors and the shear velocities for the soft-soil and the "firm to hard" rock site classes at the specified input ground motion level (Borcherdt, 1993). The equations in Figure C4.1.2-9 provide a framework to illustrate a simple procedure for derivation of amplification factors that are in general agreement with the consensus values included in Tables 1.4.2.3a and 1.4.2.3b of the *Provisions*. However, the numbers in these tables of the *Provisions* are not necessarily identical to the equations' predictions due to other considerations discussed during the consensus process.

Extensive site response studies using both equivalent linear and nonlinear programs were conducted by several groups as listed by Rinne and Dobry (1992). The main objectives of these studies were to generalize the experience of well documented earthquakes such as Loma Prieta and Mexico City to a variety of site conditions and earthquake types and levels of shaking. Some results obtained by Dobry et al. (1994a) are reproduced in Figures C4.1.2-10 to C4.1.2-12.

Figure C4.1.2-10 presents values of peak amplification at long periods for soft sites (labeled RRS_{max} in the figure) calculated using the equivalent linear approach as a function of the plasticity index (*PI*) of the soil, rock wave velocity v_r, and for weak and strong shaking. The effect of PI is due to the fact that soils with higher PI exhibit less stress-strain nonlinearity and a lower damping b_s (Vucetic and Dobry, 1991). For $S_S A_a = 0.25g$, $S_1 = 0.1g$, $v_r = 4{,}000$ ft/sec (1220 m/s) and PI = 50, roughly representative of Bay area soft sites in the Loma Prieta earthquake, $RRS_{max} = 4.4$, which coincides with the upper part of the range backfigured by Borcherdt from the records. Note the reduction of this value of RRS_{max} from 4.4 to about 3.3 when $S_S = 1.0g$, $S_1 = 0.4g$ due to soil nonlinearity. Evidence such as this is used in the 1994 *Provisions* to extrapolate values of F_a and F_v at low levels of shaking--based on both analysis and observations--to high levels of shaking for which no observations on soft sites currently are available.

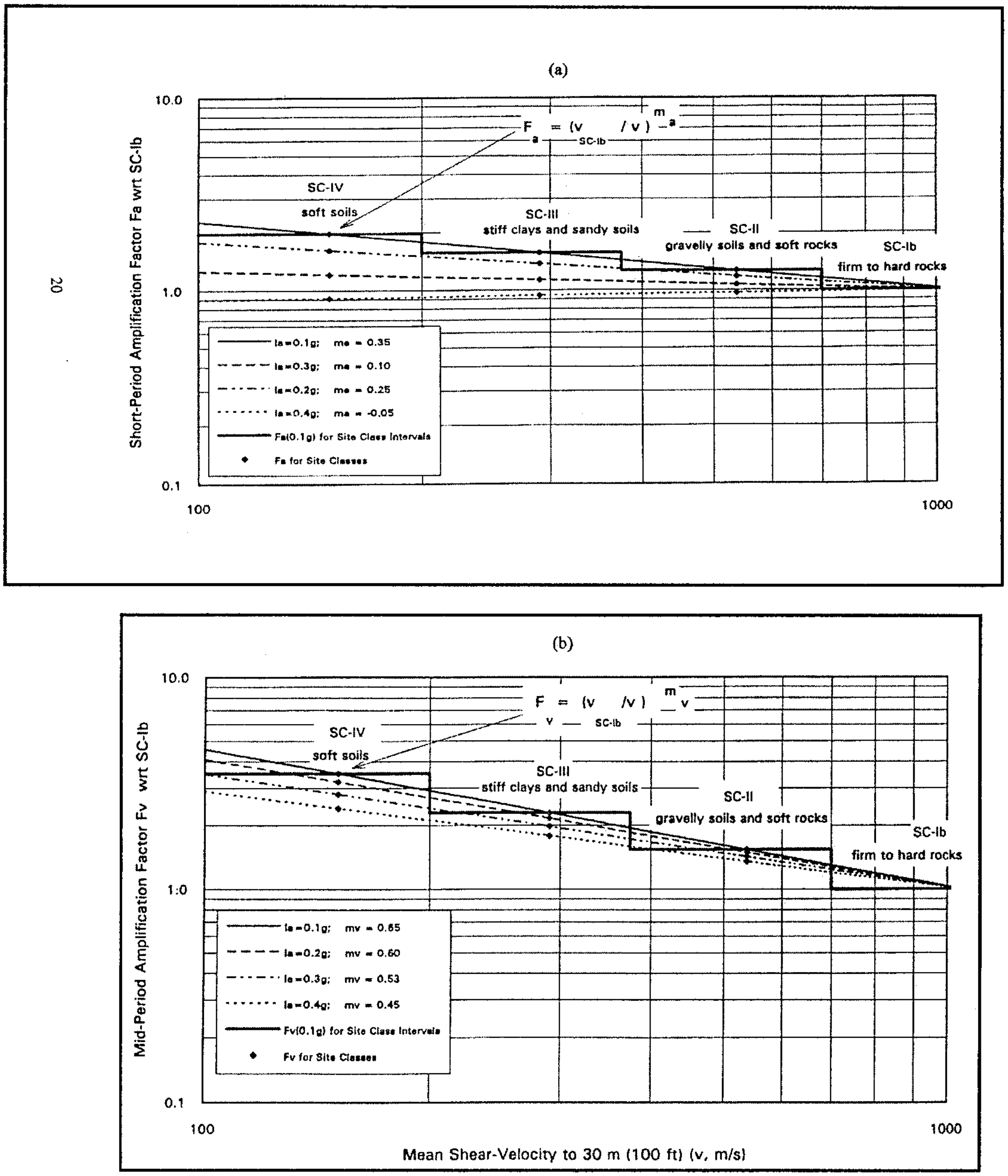


FIGURE C4.1.2-9(a) short-period F_a and (b) mid-period F_v amplification factors with respect to "firm to hard" rock (SC-Ib) plotted with logarithmic scales as a continuous function of mean shear wave velocity using the indicated equations for specified levels of input ground motion. The equations correspond to straight lines determined by the points defined as the logarithms of the amplification factors and shear velocities for the "soft-soil" and "firm to hard" rock site classes. The amplification factors for the "soft-soil" site class are based on strong motion recordings at the 0.1g level and on numerical modeling and expert opinion results for higher levels of motion. The exponents *ma* and *mv* are given by the slope of the indicated straight lines. Amplification factors with respect to SC-Ib for the amplified site classes are shown for the corresponding mean shear wave velocity interval for input ground motion levels near 0.1g (Borcherdt, 1993)

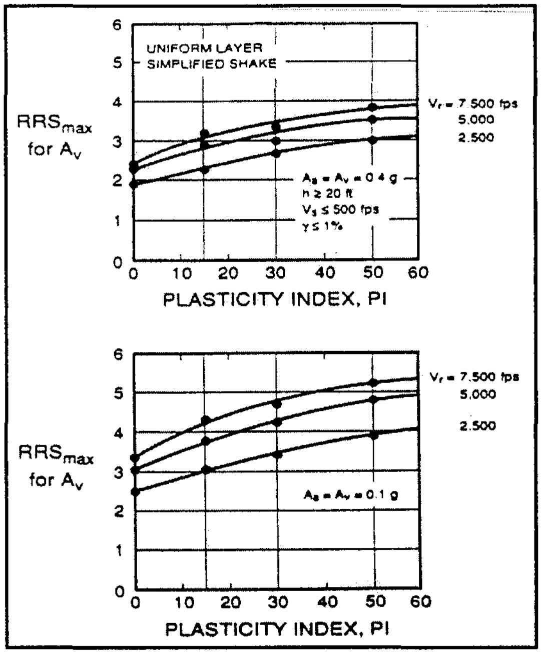


FIGURE C4.1.2-10 Summary of uniform layer analysis using simple SHAKE (Dobry et al., 1994a).

Specific equivalent linear runs using the SHAKE program corresponding to the same situation are included in Figure C4.1.2-11 while Figure C4.1.2-12 summarizes and compares them with calculations by Joyner et al. (1994) from the Loma Prieta records on soft sites similar to the work by Borcherdt mentioned above.

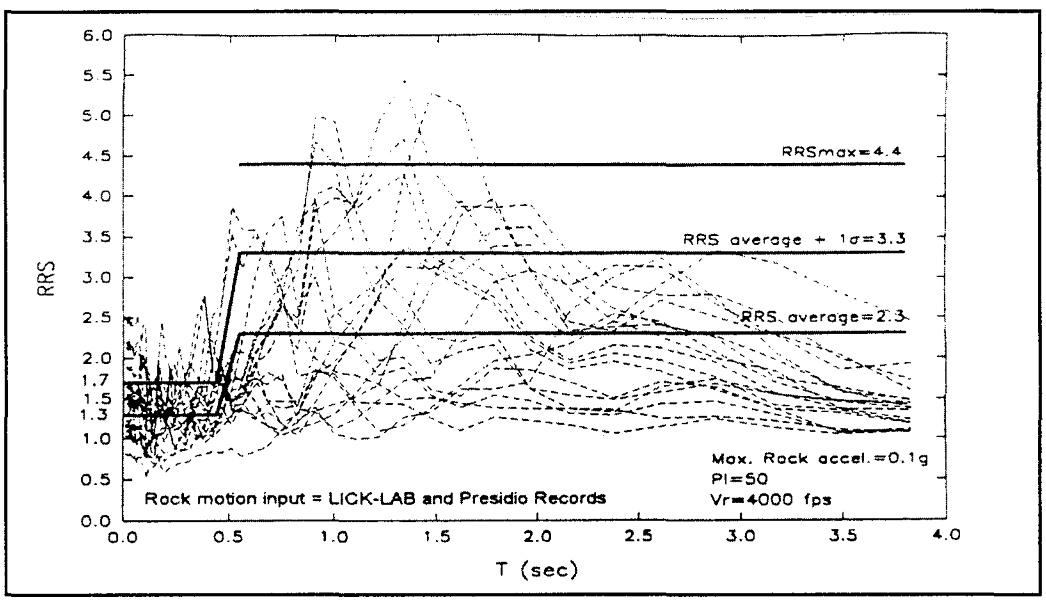


FIGURE C4.1.2-11 Summary of uniform layer analysis usinf SHAKE program, $h \geq 50$ ft (15.2m) (Dobry et al., 1994a).

Another important observation from analytical results such as shown in Figure C4.1.2-11 is that the values of RRS_{max} are about 20 percent higher for soft sites on "hard rock"--characterized by v_r = 7,500 ft/sec (2290 m/s)--than for soft sites on "regular rock" corresponding to v_r = 4,000 ft/sec (1220 m/s). This is again the impedance ratio effect previously discussed. Separate studies indicate that earthquake motions on outcrops of "hard rock" tend to be smaller than on outcrops of "regular rock" by 10 to 40 percent at both short and long periods (except at very small periods under about 0.2 sec where the reverse may be true); see Su et al. (1992) and Silva (1992). On the basis of these studies and observations, the 1994 *Provisions* incorporate the difference between "regular" rock (B) and "hard" rock of $_s$ > 5,000 ft/sec (1520 m/s) by defining a new "hard rock" site category (A) and assigning to it site factors $F_a = F_v = 0.8$.

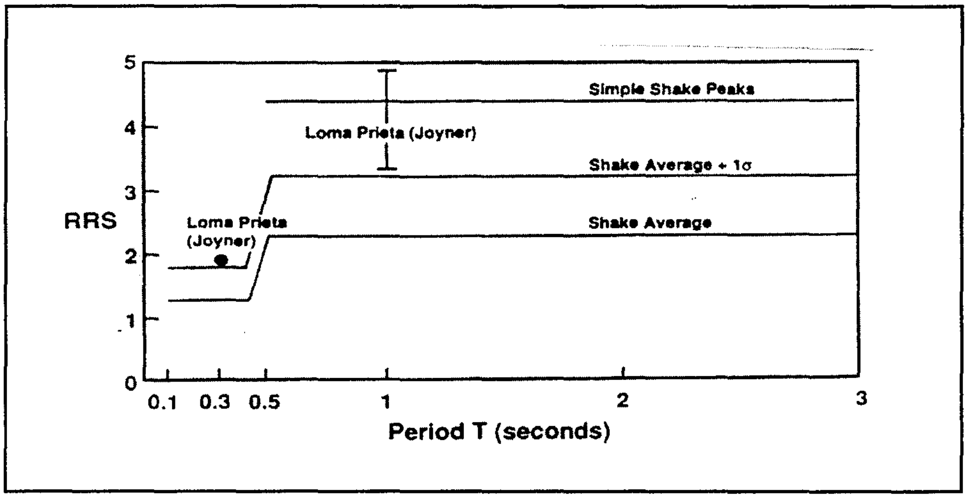


FIGURE 4.1.2-12 Comparison between RRS SHAKE program results and those obtained by Joyner et al. (1994) for the 1989 Loma Prieta event (Dobry et al., 1994a).

Use of Geotechnical Parameters Instead of v_s: Based on the studies and observations discussed above, the site categories in the 1994 *Provisions* are defined in terms of the average shear wave velocity in the top 100 ft (30.5 m) of the profile, v_s. If the shear wave velocities are available for the site, they should be used.

However, in recognition of the fact that in many cases the shear wave velocities are not available, alternative definitions of the site categories also are included in the 1994 *Provisions*. They use the standard penetration resistance for cohesionless soil layers and the undrained shear strength for cohesive soil layers. These alternative definitions are rather conservative since the correlation between site amplification and these geotechnical parameters is more uncertain than that with v_s. That is, there will be cases when the values of F_a and F_v will be smaller if the site category is based on v_s rather than on the geotechnical parameters. Also, the reader must not interpret the site category definitions as implying any specific numerical correlation between shear wave velocity on the one hand and standard penetration or shear strength on the other.

Conducting Site-Specific Geotechnical Investigations and Dynamic Site Response Analysis for Site Class F Soils: As indicated in Sec. 4.1.2.1 and in notes to Tables 4.1.2.4a and b, site coefficients F_a and F_v are not provided for Site Class F soils and site-specific geotechnical investigations and dynamic site response analyses are required for these soils. The exception is that for structures having a fundamental period of vibration equal to or less than 0.5 second, values of F_a and F_v for liquefiable soils, may be determined by following the steps for classifying a site in Sec. 4.1.2.2 assuming liquefaction does not occur. The exception is provided because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions are attenuated due to liquefaction whereas long-period ground motions may be amplified. Guidelines are provided below for conducting site-specific investigations and site response analyses for Site Class F soils. These guidelines are also applicable if it is desired to conduct dynamic site response analyses for other soil ypes.

Site-Specific Geotechnical Investigation: For purposes of obtaining data to conduct a site response analysis, site-specific geotechnical investigations should include borings with sampling, standard penetration tests (SPTs), cone penetrometer tests (CPTs), and/or other subsurface investigative techniques and laboratory soil testing to establish the soil types, properties, and layering and the depth to rock or rock-like material. It is desirable to measure shear wave velocities in all soil layers. Alternatively, shear wave velocities may be estimated based on shear wave velocity data available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Dynamic Site Response Analysis: *Components* of a dynamic site response analysis include the following steps:

1. Modeling the soil profile--Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. However, two- to three-dimensional models may be considered for critical projects when two or three-dimensional wave propagation effects may be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by their total unit weights shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress-strain relationships of the soils. The required relationships for analysis are often in the form of curves that describe the variation of shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poissons ratios also are required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model also must incorporate the buildup of soil pore water pressures and the consequent effects on reducing soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (e.g., Seed and Idriss, 1970; Seed et al., 1986; Sun et al., 1988; Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Kramer, 1996). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. The uncertainty in soil properties should be estimated, especially the uncertainty in the selected maximum shear moduli and modulus reduction and damping curves.

2. Selecting input rock motions-- Acceleration time histories that are representative of horizontal rock motions at the site are required as input to the soil model. Unless a site-specific analysis is carried out to develop the rock response spectrum at the site, the maximum considered earthquake (MCE) rock spectrum for Site Class B rock can be defined using the general procedure described in Sec. 4.1.2. For hard rock (Site Class A), the spectrum may be adjusted using the site factors in Tables 4.1.2.4a and b. For profiles having great depths of soil above Site Slass A or B rock, consideration can be given to defining the base of the soil profile and the input rock motions at a depth at which soft rock or very stiff soil of Site Class C is encountered. In such cases, the MCE rock response spectrum may be taken as the spectrum for Site Class C defined using the site factors in Tables 4.1.2.4a and b. Several acceleration time histories, typically at least four, recorded during earthquakes having magnitudes and distances that significantly contribute to the site seismic hazard should be selected for analysis. The U.S. Geological Survey results for deaggregation of seismic hazard (website address: http://geohazards.cr.usgs.gov/eq/) can be used to evaluate the dominant magnitudes and distances contributing to the hazard. Prior to analysis, each time history should be scaled so that its spectrum is at the approximate level of the MCE rock response spectrum in the period range of interest. It is desirable that the average of the response spectra of the suite of scaled input time histories be approximately at the level of the MCE rock response spectrum in the period range of interest. Because rock response spectra are defined at the ground surface rather than at depth below a soil deposit, the rock time histories should be input in the analysis as outcropping rock motions rather than at the soil-rock interface.

3. Site response analysis and results interpretation-- Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Idriss and Sun, 1992) and the nonlinear programs DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), D-MOD (Matasovic, 1993), and TESS (Pyke, 1992). For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) must be used (e.g., DESRA-2, SUMDES, D-MOD, and TESS). Response spectra of output motions at the ground surface should be calculated and the ratios of response spectra of ground surface motions to input outcropping rock motions should be calculated. Typically, an average of the response spectral ratio curves is obtained and multiplied by the MCE rock response spectrum to obtain the MCE soil design response spectrum. Sensitivity analyses to evaluate effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

4.1.2.5 Design Spectral Response Acceleration Parameters: This section provides a general method for obtaining a 5 percent damped response spectrum from the site design acceleration response parameters S_{aS} and S_{a1}. This spectrum is based on that proposed by Newmark and Hall, as a series of three curves representing in the short period, a region of constant spectral response acceleration; in the long period a range of constant spectral response velocity; and in the very long period, a range of constant spectral response displacement. Response acceleration at any period in the long period range can be related to the constant response velocity by the equation:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v \tag{C4.1.2.5-1}$$

where ω is the circular frequency of motion, T is the period and S_v is the constant spectral response velocity. The site design spectral response acceleration at 1 second, S_{a1}, therefore is simply related to the constant spectral velocity for the spectrum by the relation:

$$S_{a1} = 2\pi S_v \tag{C4.1.2.5-2}$$

and the spectral response acceleration at any period in the constant velocity range can be obtained from the relationship:

$$S_a = \frac{S_{a1}}{T} \tag{C4.1.2.5-3}$$

The constant displacement domain of the response spectrum is not included on the generalized response spectrum because relatively few structures have a period long enough to fall into this range. Response accelerations in the constant displacement domain can be related to the constant displacement by a $1/T^2$ relationship. Sec. 5.5 of the *Provisions,* which provides the requirements for modal analysis also provides instructions for obtaining response accelerations in the very long period range.

4.2 SEISMIC DESIGN CATEGORY: This section establishes the five design categories that are the keys for establishing design requirements for any building based on its use (Seismic Use Group) and on the level of expected seismic ground motion. Once the Seismic Design Category (A, B, C, D, E, or F) for the building is established, many other requirements such as detailing, quality assurance, systems and height limitations, specialized requirements, and change of use are related to it.

Prior to the 1997 edition of the *Provisions*, these categories were termed Seismic Performance Categories. While the desired performance of the building, under the design earthquake, was one consideration used to determine which category a building should be assigned to, it was not the only factor. The seismic hazard at the site was actually the principle parameter that affected a building's category. The name was changed to Seismic Design Category to represent the uses of these categories, which is to determine the specific design requirements.

The earlier editions of the *Provisions* utilized the peak velocity related acceleration, A_v, to determine a building's Seismic Performance Category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 *Provisions* adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose. Instead of a single table, as was present in previous editions of the *Provisions*, two tables are now provided, relating respectively to short period and long period structures.

Seismic Design Category A represents structures in regions where anticipated ground motions are minor, even for very long return periods. For such structures, the *Provisions* require only that a complete lateral-force-resisting system be provided and that all elements of the structure be tied together. A nominal design force of 1 percent of the weight of the structure is used to proportion the lateral system.

It is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion for Seismic Design Category A structures because the ground motion computed for the areas where these structures are located is determined more by the rarity of the event with respect to the chosen level of probability than by the level of motion that would occur if a small but close earthquake actually did occur. However, it is desirable to provide some protection against both earthquakes and many other types of unanticipated loadings. Thus, the requirements for Seismic Design Category A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible but rare earthquake even though it is possible that the ground motions could be large enough to cause serious damage or even collapse. The result of design to Seismic Design Category A requirements is that fewer building would collapse in the vicinity of such an earthquake.

The integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be identified. Then it must be designed for a lateral force equal to a 1 percent acceleration on the mass. The minimum connection forces specified for Seismic Design Category A also must be satisfied.

The 1 percent value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local buildings codes normally will control the lateral force design when compared to the minimum integrity force on the structure. However, many low-rise, heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1 percent acceleration. Also, minimum connection forces may exceed structural forces due to wind in some structures.

Seismic Design Category B includes Seismic Use Group I and II structures is regions of seismicity where only moderately destructive ground shaking is anticipated. In addition to the requirements for Seismic Design Category A, structures in Seismic Design Category B must be designed for forces determined using Maps 1 through 24.

Seismic Design Category C includes Seismic Use Group III structures in regions where moderately destructive ground shaking may occur as well as Seismic Use Group I and II structures in regions with somewhat more severe ground shaking potential. In Seismic Design Category C, the use of some structural systems is limited and some nonstructural *components* must be specifically design for seismic resistance.

Seismic Design Category D includes structures of Seismic Use Group I, II, and III located in regions expected to experience destructive ground shaking but not located very near major active faults. In Seismic Design Category D, severe limits are placed on the use of some structural systems and irregular structures must be subjected to dynamic analysis techniques as part of the design process.

Seismic Design Category E includes Seismic Use Group I and II structures in regions located very close to major active faults and Seismic Design Category F includes Seismic Use Group III structures in these locations. Very severe limitations on systems, irregularities, and design methods are specified for Seismic Design Categories E and F. For the purpose of determining if a structure is located in a region that is very close to a major active fault, the *Provisions* use a trigger of a mapped maximum considered earthquake spectral response acceleration at 1 second periods, S_1, of 0.75g or more regardless of the structure's fundamental period. The mapped short period acceleration, S_S, was not used for this purpose because short period response accelerations do not tend to be affected by near-source conditions as strongly as do response accelerations at longer periods.

Local or regional jurisdictions enforcing building regulations need to consider the effect of the maps, typical soil conditions, and Seismic Design Categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular Site Classes, or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example:

1. An area with an historical practice of high seismic zone detailing might mandate a minimum Seismic Design Category of D regardless of ground motion or Site Class.

2. A jurisdiction with low variation in ground motion across the area might stipulate particular values of the ground motion rather than requiring use of the maps.

3. An area with unusual soils might require use of a particular Site Class unless a geotechnical investigation proves a better Site Class.

4.2.2 Site Limitation for Seismic Design Categories E and F: The forces that result on a structure located astride the trace of a fault rupture that propagates to the surface are extremely large and it is not possibly to reliably design a structure to resist such forces. Consequently, the requirements of this section limit the construction of buildings in Seismic Design Categories E and F on sites subject this hazard. Similarly, the effects of landsliding, liquefaction, and lateral spreading can be highly damaging to a building. However, the effects of these site phenomena can more readily be mitigated through the incorporation of appropriate design measures than can direct ground fault rupture. Consequently, construction on sites with these hazards is permitted, if appropriate mitigation measures are included in the design.

REFERENCES

Borcherdt, R. D. 1994. "Simplified Site Classes and Empirical Amplification Factors for Site-Dependent Code Provisions." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Borcherdt, R. D. 1993. "General Methodology for Estimating Free-Field Site Specific Response Spectra." In Minutes of the Building Seismic Safety Council meeting, April 5-6, 1993, San Francisco, California.

Borcherdt, R. D., and G. Glassmoyer. 1993. "Influence of Local Geology and Weak Ground Motions in the San Francisco Bay Region, California, and Their Implications for Site-Specific Code Provisions." In *The Loma Prieta Earthquake of October 17, 1989 - Strong Ground Motion*, USGS Professional Paper 1551-A, edited by R. D. Borcherdt.

Chang, C.-Y., C.M. Mok,, M.S. Power, and Y.K. Tang, 1991. *Analysis of Ground Response at Lotung Large-Scale Soil-Structure Interaction Experiment Site*, Report NP-7306-SL. Palo Alto, California: Electric Power Research Institute.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994a. "Development of Site-Dependent Ratios of Elastic Response Spectra (RRS) and Site Categories for Building Seismic Codes." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994b. *Study of Ratios of Response Spectra Soil/Rock and of Site Categories for Seismic Codes*. Buffalo, New York: National Center for Earthquake Engineering Research.

Electric Power Research Institute. 1993. *Guidelines for Determining Design Basis Ground Motions*, Report EPRI TR-102293. Palo Alto, California.

Hayashi, S., Tsuchida, and E. Kurata. 1971. "Average Response Spectra for Various Subsoil Conditions." Paper presented at the Third Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects, UNJR, Tokyo, May 10-12.

Housner, G. W. 1990. *Competing Against Time: Report to Governor George Deukmejian from the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake.*

Idriss, I. M. 1990a. "Response of Soft Soil Sites During Earthquakes." In *Proceedings of the Symposium to Honor Professor H. B. Seed*, Berkeley.

Idriss, I. M. 1990b. "Influence of Local Site Conditions on Earthquake Ground Motions." In *Proceedings of the 4th U.S. National Conference on Earthquake Engineering*, Palm Springs, Vol. 1, pp. 55-57.

Idriss, I. M., and J. I.Sun.1992. *User's Manual for SHAKE91.* Davis: Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California.

Jacob, K. 1990. "Seismic Hazards and the Effects of Soils on Ground Motions for the Greater New York City Metropolitan Region." In *Geotechnical Aspects of Seismic Design in the N.Y.C. Metropolitan Areas; Risk Assessment, Code Requirements and Design Techniques*, ASCE Metropolitan Section, page 24, New York, New York, November 13-14.

Jacob, K. 1991. "Seismic Zonation and Site Response: Are Building-Code Soil Factors Adequate to Account for Variability of Soil Conditions Across the US?" In *Proceedings of the 4th International Conference on Seismic Zonation*, Vol. II, pp. 695-702. Oakland, California: Earthquake Engineering Research Institute.

Joyner, W. B., T. E. Fumal, and G. Glassmoyer. 1994. "Empirical Spectral Response Ratios for Strong Motion Data from the 1989 Loma Prieta, California, Earthquake." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Kramer, S. L., 1996. *Geotechnical Earthquake Engineering.* New Jersey: Prentice Hall.

Lee, M. K. W., and W. D. L. Finn. 1978. *DESRA-2, Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential, Soil Mechanics Series 36.* Vancouver, Canada: Department of Civil Engineering, University of British Columbia.

Li, X. S., Z. L. Wang, and C. K. Shen, 1992. *SUMDES, A Nonlinear Procedure for Response Analysis of Horizontally-Layered Sites Subjected to Multi-Directional Earthquake Loading.* Davis: Department of Civil Engineering, University of California.

Martin, G. M., Editor. 1994. *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20.

Matasovic, N. 1993. Seismic Response of Composite Horizontally-Layered Soil Deposits, Ph.D. dissertation, Civil and Environmental Engineering Department, University of California, Los Angeles.

Milne, W. G., and A. G. Davenport. 1969. "Distribution of Earthquake Risk in Canada." *Bulletin of the Seismological Society of America* 59(2):754-779.

Mohraz, B. 1976. "A Study of Earthquake Response Spectra for Different Geological Conditions." *Bulletin of the Seismological Society of America* 66 (3):915-935.

Newmark, N. M. 1973. *A Study of Vertical and Horizontal Spectra,* Report WASH-1255. Washington, D.C.: U.S. Atomic Energy Commission, Directorate of Licensing.

Ordaz, M., and Arciniegas. 1992. Personal communication to Ricardo Dobry.

Pyke, R. M. 1992, *TESS: A Computer Program for Nonlinear Ground Response Analyses.* Lafayette, California: TAGA Engineering Systems and Software.

Rinne, E., and R. Dobry. 1992. Preliminary Site Recommendations, Memorandum to Roland Sharpe, Chairman TS 2, Building Seismic Safety Council, December 11.

Roesset, J. M. 1977. "Soil Amplification of Earthquakes." In *Numerical Methods in Geotechnical Engineering*, edited by C. S. Desai and J. T. Christian, Chapter 19, pp. 639-682. New York: McGraw-Hill.

Seed, R. B. 1994. In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Seed, R. B., S. E. Dickenson, G. A. Rau, R. K. White, and C. M. Mok. 1992. "Observations Regarding Seismic Response Analyses for Soft and Deep Clay Sites."

Seed, R., S. E. Dickenson, M. F. Riemer, J. D. Bray, N. Sitar, J. K. Mitchell, I. M. Idriss, R. E. Kayen, A. Kropp, L. F. Harder Jr., and M. S. Power. 1990. *Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989, Loma Prieta Earthquake.* Report UCB/EERC-90/05. Berkeley, California: EERC.

Seed, H. B., R. Murarka, J. Lysmer, and I. M. Idriss. 1976a. "Relationships Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong Earthquakes." *Bulletin of the Seismological Society of America* 66 (4):1323--1342.

Seed, H. B., M. P. Romo, J. I. Sun, A. Jaime, and J. Lysmer. 1988. "The Mexico Earthquake of September 19, 1985--Relationships Between Soil Conditions and Earthquake Ground Motions." *Earthquake Spectra* 4(4):687-729.

Seed, H. B., C. Ugas, and J. Lysmer. 1976b. "Site Dependent Spectra for Earthquake-Resistant Design." *Bulletin of the Seismological Society of America* 66 (1):221-244.

Seed, H. B., C. Ugas, and J. Lysmer. 1974. *Site-Dependent Spectra for Earthquake-Resistant Design*, Report EERC 74-12. Berkeley, California: EERC.

Seed, H.B., R. T. Wong, I. M. Idriss, and K. Tokimatsu. 1986. "*Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils*", Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 11: 1016-1032.

Seed, H.B., and I. M. Idriss. 1970, *Soil Moduli and Damping Factors for Dynamic Response Analyses*, Report EERC 70-10. Berkeley: University of California, Earthquake Engineering Research Center.

Silva, W. 1992. Personal communication to Ricardo Dobry.

Structural Engineers Association of California. 1970. *Report of the Ad Hoc Committee on Cost of Design for Earthquakes.* Washington, D.C.: Office of Science and Technology, Task Force on Earthquake Hazard Reduction.

Su, F., K. Aki, T. Teng, Y. Zeng, S. Koyanagi, and K. Mayeda. 1992. "The Relation Between Site Amplification Factor and Surficial Geology in Central California." *Bulletin of the Seismological Society of America* 82(2):580-602.

Sun, J. I., R. Golesorkhi, and H. B. Seed. 1988. *Dynamic Moduli and Damping Ratios for Cohesive Soils*, Report UBC/EERC-88/15. Berkeley: University of California, Earthquake Engineering Research Center.

Vucetic, M., and R. Dobry. 1991. "*Effect of Soil Plasticity on Cyclic Response*", Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 1: 89-107.

Whitman, R., Editor. 1992. *Proceedings of the Site Effects Workshop, October 24-24, 1991*, Report NCEER-92-0006. Buffalo, New York: National Center for Earthquake Engineering Research.

Chapter 5 Commentary

STRUCTURAL DESIGN CRITERIA

5.1 REFERENCE DOCUMENT: ASCE 7 is referenced for the combination of earthquake loadings with other loads as well as for the computation of other loads; it is not referenced for the computation of earthquake loads.

5.2 DESIGN BASIS: Structural design for acceptable seismic resistance includes:

1. The selection of vertical and lateral-force-resisting systems that are appropriate to the anticipated intensity of ground shaking;
2. Layout of these systems such that they provide a continuous, regular and redundant load path capable of ensuring that the structures act as integral units in responding to ground shaking; and
3. Proportioning the various members and connections such that adequate lateral and vertical strength and stiffness is present to limit damage in a design earthquake to acceptable levels.

In the *Provisions*, the proportioning of structures' elements (sizing of individual members, connections, and supports) is typically based on the distribution of internal forces computed based on linear elastic response spectrum analyses using response spectra that are representative of, but substantially reduced from the anticipated design ground motions. As a result, under the severe levels of ground shaking anticipated for many regions of the nation, the internal forces and deformations produced in most structures will substantially exceed the point at which elements of the structures start to yield and buckle and behave in an inelastic manner. This approach can be taken because historical precedent, and the observation of the behavior of structures that have been subjected to earthquakes in the past demonstrates that if suitable structural systems are selected, and structures are detailed with appropriate levels of ductility, regularity, and continuity, it is possible to perform an elastic design of structures for reduced forces and still achieve acceptable performance. Therefore, these procedures adopt the approach of proportioning structures such that under prescribed design lateral forces that are significantly reduced, by the response modification coefficient R, from those that would actually be produced by a design earthquake they will not deform beyond a point of significant yield. The elastic deformations calculated under these reduced design forces are then amplified, by the deflection amplification factor C_d to estimate the expected deformations likely to be experienced in response to the design ground motion. (The deflection amplification is specified in Sec. 5.4.6.) Considering the intended structural performance and acceptable deformation levels, Sec. 5.2.8 prescribes the story drift limits for the expected (i.e. amplified) deformations. These procedures differ from those in earlier codes and design provisions wherein the drift limits were treated as a serviceability check.

The term "significant yield" is not the point where first yield occurs in any member but, rather, is defined as that level causing complete plastification of at least the most critical region of the structure (e.g., formation of a first plastic hinge in the structure). A structural steel frame comprised of compact members is assumed to reach this point when a "plastic hinge" develops in the most highly stressed member of the structure. A concrete frame reaches this significant yield when at least one of the sections of its most highly stressed *component* reaches its strength as set forth in Chapter 9. For other structural materials that do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided. These requirements contemplate that the design includes a seismic force resisting system with redundant characteristics wherein significant structural overstrength above the level of significant yield can be obtained by plastification at other points in the structure prior to the formation of a complete mechanism. For example, Figure C5.2-1 shows the lateral load-deflection curve for a typical structure. Significant yield is the level where plastification occurs at the most heavily loaded element in the structure, shown as the lowest yield hinge on the load-deflection diagram. With increased loading, causing the formation of additional plastic hinges, the capacity increases (following the solid curve) until a maximum is reached. The overstrength capacity obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the design ground motion.

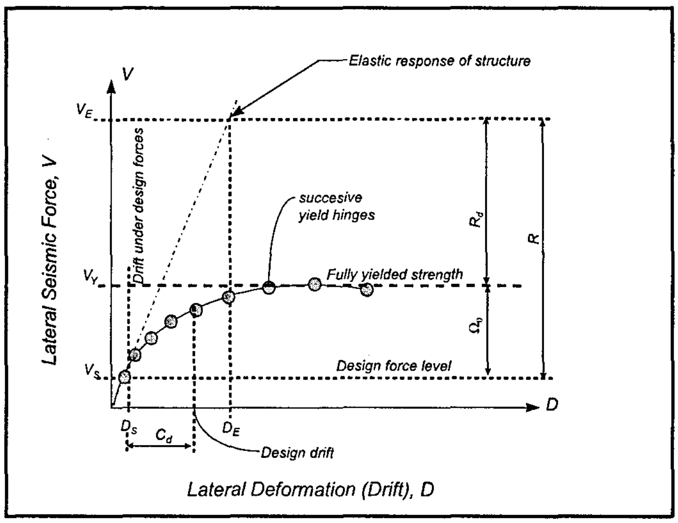


FIGURE C5.2-1 Inelastic force-deformation curve.

It should be noted that the structural overstrength described above results from the development of sequential plastic hinging in a properly designed, redundant structure. Several other sources will further increase structural overstrength. First, material overstrength (i.e. actual material strengths higher than the nominal material strengths specified in the design) may increase the structural overstrength significantly. For example, a recent survey shows that the mean yield strength of A36 steel is about 30 to 40 percent higher than the minimum specified strength, nominally used in design calculations. Second, member design strengths usually incorporate a strength reduction (or resistance) factor, ϕ, to ensure a low probability of failure under design loading. Third, designers themselves introduce additional overstrength by selecting sections or specifying reinforcing patterns that exceed those required by the computations. Similar situations occur when minimum requirements of the *Provisions*, for example, minimum reinforcement ratios, control the design. Finally, the design of many flexible structural systems, such as moment resisting frames, are often controlled by the drift rather than strength limitations of the *Provisions*, with sections selected to control lateral deformations rather than provide the specified strength. The results is that structures typically have a much higher lateral resistance than specified as a minimum by the *Provisions* and first actual significant yielding of structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design seismic forces. If provided with adequate ductile detailing, redundancy and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels.

Figure C5.2-1 indicates the significance of design parameters contained in the *Provisions* including the response modification coefficient, *R*, the deflection amplification factor, $C_{d,}$ and the structural overstrength coefficient Ω_0. The values of the response modification coefficient, *R*, structural overstrength coefficient, Ω_0, and the deflection amplification factor, $C_{d,}$ provided in Table 5.2.2, as well as the criteria for story drift including *P*-delta effects have been established considering the characteristics of typical properly designed structures. If excessive "optimization" of a structural design is performed, with lateral resistance provided by only a few elements, the successive yield hinge behavior depicted in Figure C5.2-1 will not be able to form and the values of the design parameters contained in the *Provisions* may not be adequate to provide the intended seismic performance.

The response modification coefficient, *R,* essentially represents the ratio of the forces that would develop under the specified ground motion if the structure had an entirely linearly elastic response to the prescribed design forces (see Figure C5.2-1). The structure is to be designed so that the level of significant yield exceeds the prescribed design force. The ratio *R,* expressed by the equation:

$$R = \frac{V_E}{V_S} \qquad \text{(C5.2.1-1)}$$

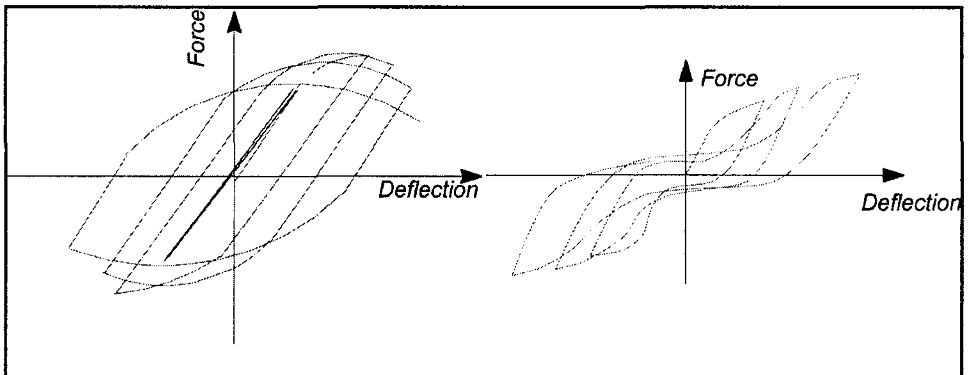
is always larger then 1.0; thus, all structures are designed for forces smaller than those the design ground motion would produce in a completely linear-elastic responding structure. This reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of response of the structure tends to lengthen, which for many structures, results in a reduction in strength demand. Furthermore, the inelastic action results in a significant amount of energy dissipation, also known as hysteretic damping, in addition to the viscous damping. The combined effect, which is also known as the ductility reduction, explains why a properly designed structure with a fully yielded strength (V_y, in Figure C.5.2-1) that is significantly lower than the elastic seismic force demand (V_E in Figure C.5.2.1) can be capable of providing satisfactory performance under the design ground motion excitations. Defining a system ductility reduction factor R_d as the ratio between V_E and V_Y (Newmark and Hall, 1981):

$$R_d = \frac{V_E}{V_Y} \qquad \text{(C5.2.1-2)}$$

then it is clear from Figure C5.2-1 that the response modification coefficient, *R*, is the product of the ductility reduction factor and structural overstrength factor (Uang, 1991):

$$R = R_d \Omega_0 \qquad \text{(C5.2.1-3)}$$

The energy dissipation resulting from hysteretic behavior can be measured as the area enclosed by the force-deformation curve of the structure as it experiences several cycles of excitation. Some structures have far more energy dissipation capacity than do others. The extent of energy dissipation capacity available is largely dependent on the amount of stiffness and strength degradation the structure undergoes as it experiences repeated cycles of inelastic deformation. Figure C5.2-2 indicates representative load-deformation curves for two simple substructures, such as a beam-column assembly in a frame. Hysteretic curve (a) in the figure is representative of the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain nearly all of its strength and stiffness over a number of large cycles of inelastic deformation. The resulting force-deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation capacity. Hysteretic curve (b) represents the behavior of a substructure that has not been detailed for ductile behavior. It rapidly looses stiffness under inelastic deformation and the resulting hysteretic loops are quite pinched. The energy dissipation capacity of such a substructure is much lower than that for the substructure (a). Structural systems with large energy dissipation capacity have larger R_d values, and hence are assigned higher R values, resulting in design for lower forces, than systems with relatively limited energy dissipation capacity.



a. Ductile hysteris loops b. Pinched hysteris loops

FIGURE C5.2-2 Typical hysteretic curves.

Some contemporary building codes, including those adopted in Canada and Europe have attempted to directly quantify the relative contribution of overstrength and inelastic behavior to the permissible reduction in design strength. Recently, the Structural Engineers Association of California proposed such an approach for incorporation into the 1997 *Uniform Building Code.* That proposal incorporated two R factor *components*, termed R_o and R_d to represent the reduction due to structural overstrength and inelastic behavior, respectively. The design forces are then determined by forming a composite R, equal to the product of the two *components* (See Eq. C5.2.1-3). A similar approach was considered for adoption into the 1997 *NEHRP Provisions.* However, this approach was not taken for several reasons. While it was acknowledged that both structural overstrength and inelastic behavior are important contributors to the R coefficients, and can be quantified for individual structures, it was felt that there was insufficient research available at the current time to support implementation in the *Provisions*. In addition, there was concern that there can be significant variation between structures in the relative contribution of overstrength and inelastic behavior and that, therefore, this would prevent accurate quantification on a system by system basis. Finally, it was felt that this would introduce additional complexity into the *Provisions*. While it was decided not to introduce the split *R* value concept into the *Provisions* in the 1997 update cycle, this should be considered in the future as additional research on the inelastic behavior of structures becomes available, and as the sophistication of design offices improves to the point that quantification of structural overstrength can be done as a routine part of the design process. As a first step in this direction, however, the factor Ω_0 was added to Table 5.2.2, to replace the previous $2R/5$ factor used for evaluation of brittle structural behavior modes in previous editions of the *Provisions*.

The *R* values, contained in the current *Provisions*, are largely based on engineering judgment of the performance of the various materials and systems in past earthquakes. The values of *R* must be chosen and used with careful judgment. For example, lower values must be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental *P*-delta effects. Since it is difficult for individual designers to judge the extent to which *R* factors should be adjusted, based on the inherent redundancy of their designs, a new coefficient ρ, that is calculated based on percent of the total lateral force resisted by any individual element has been introduced into the *Provisions* in Sec. 5.2.4. Additional discussion of this issue is contained in that section.

In a departure from previous editions of the *Provisions*, the 1997 edition introduced an importance factor I into the base shear equation, that varies for different types of occupancies. This importance factor has the effect of adjusting the permissible response modification factor, *R*, based on the desired seismic performance for the structure. It recognizes that as structures experience greater levels of inelastic behavior, they also experience more damage. Thus, introducing the importance factor, *I*, allows for a reduction of the *R* value to an effective value R/I as a partial control on the amount of damage experienced by the structure under a design earthquake. Strength alone is not sufficient to obtain enhanced seismic performance. Therefore, the improved performance characteristics desired for more critical occupancies are also obtained through application of the design and detailing requirements set forth in Sec. 5.2.6 for each Seismic Design Category and the more stringent drift limits in Table 5.2.8. These factors, in addition to strength, are extremely important to obtaining the seismic performance desired for buildings in some Seismic Use Groups.

Sec. 5.2.1 in effect calls for the seismic design to be complete and in accordance with the principles of structural mechanics. The loads must be transferred rationally from their point of origin to the final points of resistance. This should be obvious but it often is overlooked by those inexperienced in earthquake engineering.

5.2.2 Basic Seismic-Force-Resisting Systems: For purposes of these seismic analyses and design requirements, building framing systems are grouped in the structural system categories shown in Table 5.2.2. These categories are similar to those contained for many years in the requirements of the *Uniform Building Code;* however, a further breakdown is included for the various types of vertical *components* in the seismic-force-resisting system. In selecting a structural system, the designer is cautioned to consider carefully the interrelationship between continuity, toughness (including minimizing brittle behavior), and redundancy in the structural framing system as is subsequently discussed in this commentary.

Specification of R factors requires considerable judgment based on knowledge of actual earthquake performance as well as research studies; yet, they have a major effect on building costs. The factors in Table 5.2.2 continue to be reviewed in light of recent research results. In the selection of the R values for the various systems, consideration has been given to the general observed performance of each of the system types during past earthquakes, the general toughness (ability to dissipate energy without serious degradation) of the system, and the general amount of damping present in the system when undergoing inelastic response. The designer is cautioned to be especially careful in detailing the more brittle types of systems (low C_d values).

A bearing wall system refers to that structural support system wherein major load-carrying columns are omitted and the walls and/or partitions are of sufficient strength to carry the gravity loads for some portion of the building (including live loads, floors, roofs, and the weight of the walls themselves). The walls and partitions supply, in plane, lateral stiffness and stability to resist wind and earthquake loadings as well as any other lateral loads. In some cases, vertical trusses are employed to augment lateral stiffness. In general, this system has comparably lower values of R than the other systems due to the frequent lack of redundancy for the vertical and horizontal load support. The category designated "light frame walls with shear panels" is intended to cover wood or steel stud wall systems with finishes other than masonry veneers.

A building frame system is a system in which the gravity loads are carried primarily by a frame supported on columns rather than by bearing walls. Some minor portions of the gravity load may be carried on bearing walls but the amount so carried should not represent more than a few percent of the building area. Lateral resistance is provided by nonbearing structural walls or braced frames. The light frame walls with shear panels are intended only for use with wood and steel building frames. Although there is no requirement to provide lateral resistance in this framing system, it is strongly recommended that some moment resistance be incorporated at the joints. In a structural steel frame, this could be in the form of top and bottom clip angles or tees at the beam- or girder-to-column connections. In reinforced concrete, continuity and full anchorage of longitudinal steel and stirrups over the length of beams and girders framing into columns would be a good design practice. With this type of interconnection, the frame becomes capable of providing a nominal secondary line of resistance even though the *components* of the seismic-force-resisting system are designed to carry all the seismic force.

A moment resisting space frame system is a system having an essentially complete space frame as in the building frame system. However, in this system, the primary lateral resistance is provided by moment resisting frames composed of columns with interacting beams or girders. Moment resisting frames may be either ordinary, intermediate, or special moment frames as indicated in Table 5.2.2 and limited by the Seismic Design Categories.

Special moment frames must meet all the design and detail requirements of Chapter 8, 9, or 10. The ductility requirements for these frame systems are appropriate for all structures anticipated to experience large inelastic demands. For this reason, they are required in zones of high seismicity with large anticipated ground shaking accelerations. In zones of lower seismicity, the inherent overstrength in typical structural designs is such that the anticipated inelastic demands are somewhat reduced, and less ductile systems may be safely employed. For buildings in which these special design and detailing requirements are not used, lower R values are specified indicating that ordinary framing systems do not possess as much toughness and that less reduction from the elastic response can be tolerated. Note that Sec. 5.2.2 (Table 5.2.2) requires moment frames in Categories D and E or F greater than 160 ft and 100 ft in height, respectively, to be special moment frames.

Requirements for composite steel-concrete systems were first introduced in the 1994 Edition. The R, Ω_0, and C_d values for the composite systems in Table 5.2.2 are similar to those for comparable systems of structural steel and reinforced concrete. The values shown in Table 5.2.2 are only allowed when the design and detailing requirements for composite structures in Chapter 10 are followed.

Inverted pendulum structures are singled out for special consideration because of their unique characteristics. These structures have little redundancy and overstrength and concentrate inelastic behavior at their bases. As a result, they have substantially less energy dissipation capacity than other systems. A number of buildings incorporating this system experienced very severe damage, and in some cases, collapse, in the 1994 Northridge earthquake.

5.2.2.1 Dual System: A dual system consists of a three-dimensional space frame made up of columns and beams that provide primary support for the gravity loads. Primary lateral resistance is supplied by structural nonbearing walls or bracing; the frame is provided with a redundant lateral-force-resisting system that is a moment frame complying with the requirements of Chapters 8, 9, or 10. The moment frame is required to be capable of resisting at least 25 percent (judgmentally selected) of the specified seismic force. Normally the moment frame would be a part of the basic space frame. The walls or bracing acting together with the moment frame must be capable of resisting all of the design seismic force. The following analyses are required for dual systems:

1. The frame and shear walls or braced frames must resist the prescribed lateral seismic force in accordance with their relative rigidities considering fully the interaction of the walls or braced frames and the moment frames as a single system. This analysis must be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the *moment* frame by their interaction with the shear walls or braced frames must be considered in this analysis.
2. The moment frame must be designed to have a capacity to resist at least 25 percent of the total required lateral seismic force including torsional effects.

5.2.2.2 Combinations of Framing Systems: For those cases where combinations of structural systems are employed, the designer must use judgment in selecting appropriate R, Ω_0, and C_d values. The intent of Sec. 5.2.2.2.1 is to prohibit support of one system by another possessing characteristics that result in a lower base shear factor. The entire system should be designed for the higher seismic shear as the provision stipulates. The exception is included to permit the use of such systems as a braced frame penthouse on a moment frame building in which the mass of the penthouse does not represent a significant portion of the total building and, thus, would not materially affect the overall response to earthquake motions.

Sec. 5.2.2.2.2 pertains to details and is included to help ensure that the more ductile details inherent with the design for the higher R value system will be employed throughout. The intent is that details common to both systems be designed to remain functional throughout the response in order to preserve the integrity of the seismic-force-resisting system.

5.2.2.3 - 5.2.2.6 Seismic Design Categories : General framing system requirements for the building Seismic Design Categories are given in these sections. The corresponding design and detailing requirements are given in Sec. 5.2.6 and Chapters 8 through 14. Any type of building framing system permitted by the *Provisions* may be used for Categories A, B, and C except frames limited to Category A or Categories A and B only by the requirements of Chapters 9 and 12. Limitations regarding the use of different structural systems are given for Categories D, E and F.

5.2.2.4 Seismic Design Categories D and E: Sec. 5.2.2.4 covers Categories D and E, which compares roughly to California design practice for normal buildings other than hospitals. According to the requirements of Chapters 8 and 9, all moment-resisting frames of steel or concrete must be special moment frames. Note that present SEAOC and *UBC* recommendations have similar requirements for concrete frames; however, ordinary moment frames of structural steel may be used for heights up to 160 ft (49 m). In keeping with the philosophy of present codes for zones of high seismic risk, these requirements continue limitations on the use of certain types of structures over 160 ft (49 m) in height but with some changes. Although it is agreed that the lack of reliable data on the behavior of high-rise buildings whose structural systems involve shear walls and/or braced frames makes it convenient at present to establish some limits, the values of 160 ft (49 m) and 240 ft (73 m) introduced in these requirements are arbitrary. Considerable disagreement exists regarding the adequacy of these values, and it is intended that these limitations be the subject of further study.

These requirements require that buildings in Category D over 160 ft (49 m) in height have one of the following seismic-force-resisting systems:

1. A moment resisting frame system with special moment frames capable of resisting the total prescribed seismic force. This requirement is the same as present SEAOC and *UBC* recommendations.

2. A dual system as defined in the Glossary, wherein the prescribed forces are resisted by the entire system and the special moment frame is designed to resist at least 25 percent of the prescribed seismic force. This requirement is also similar to SEAOC and *UBC* recommendations. The purpose of the 25 percent frame is to provide a secondary defense system with higher degrees of redundancy and ductility in order to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. It should be noted that SEAOC and *UBC* requirements prior to 1987 required that shear walls or braced frames be able to resist the total required seismic lateral forces independently of the special moment frame. The *Provisions* require only that the true interaction behavior of the frame-shear wall (or braced frame) system be considered (see Table 5.2.2). If the analysis of the interacting behavior is based only on the seismic lateral force vertical distribution recommended in the equivalent lateral force procedure of Sec. 5.3, the interpretation of the results of this analysis for designing the shear walls or braced frame should recognize the effects of higher modes of vibration. The internal forces that can be developed in the shear walls in the upper stories can be more severe than those obtained from such analysis.

3. The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 ft (73 m) is permitted only if braced frames or shear walls in any plane do not resist more than 50 percent of the seismic design force including torsional effects and the configuration of the lateral-force-resisting system is such that torsional effects result in less than a 20 percent contribution to the strength demand on the walls or frames. The intent is that each of these shear walls or braced frames be in a different plane and that the four or more planes required be spaced adequately throughout the plan or on the perimeter of the building in such a way that the premature failure of one of the single walls or frames will not lead to excessive inelastic torsion.

Although a structural system with lateral force resistance concentrated in the interior core (Figure C5.2.2.4-1) is acceptable according to the *Provisions*, it is highly recommended that use of such a system be avoided, particularly for taller buildings. The intent is to replace it by the system with lateral force resistance distributed across the entire building (Figure C5.2.2.4-2). The latter system is believed to be more suitable in view of the lack of reliable data regarding the behavior of tall buildings having structural systems based on central cores formed by coupling shear walls or slender braced frames.

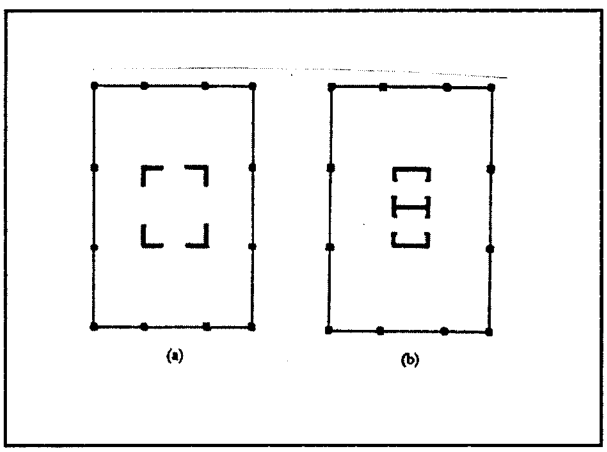


Figure C5.2.2.4-1 Arrangement of shear walls and braced frames – not recommended. Note that the heavy lines indicate shear walls and/or braced frames.

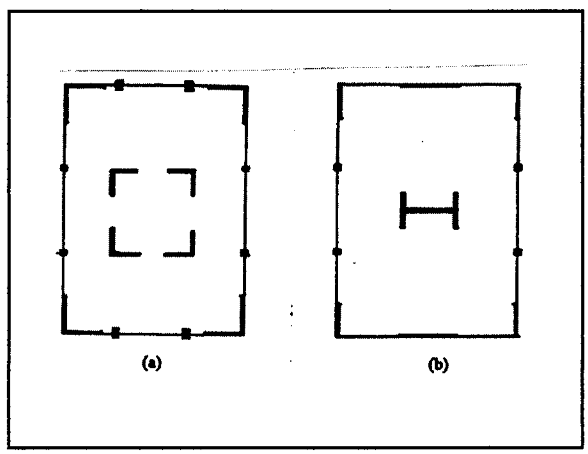


Figure C5.2.2.4-2 Arrangement of shear walls and braced frames – recommended. Note that the heavy lines indicate shear walls and/or braced frames.

5.2.2.4.2 Interaction Effects: This section relates to the interaction of elements of the seismic-force-resisting system with elements that are not part of this system. A classic example of such interaction is the behavior of infill masonry walls used as architectural elements in a building provided with a seismic-force-resisting system composed of moment resisting frames. Although the masonry walls are not intended to resist seismic forces, at low levels of deformation they will be substantially more rigid than the moment resisting frames and will participate in lateral force resistance. A common effect of such walls is that they can create shear-critical conditions in the columns they infill against by reducing the effective flexural height of these columns to the height of the openings in the walls. If these walls are not uniformly distributed throughout the structure, or not effectively isolated from participation in lateral force resistance they can also create torsional irregularities and soft story irregularities in structures that would otherwise have regular configuration.

Infill walls are not the only elements not included in seismic-force-resisting systems that can affect a structure's seismic behavior. For example, in parking garage structures, the ramps between levels can act as effective bracing elements and resist a large portion of the seismic induced forces. They can induce large thrusts in the diaphragms where they connect, as well as large vertical forces on the adjacent columns and beams. In addition, if not symmetrically placed in the structure they can induce torisional irregularities. This section requires consideration of these potential effects.

5.2.2.4.3 Deformational Compatibility: The purpose of this section is to require that the seismic-force-resisting system provide adequate deformation control to protect elements of the structure that are not part of the seismic-force-resisting system. In regions of high seismicity, it is relatively common to apply ductile detailing requirements to elements which are intended to resist seismic forces but to neglect such practices in nonstructural elements or elements intended to only resist gravity forces. The fact that many elements of the structure are not intended to resist seismic forces and are not detailed for such resistance does not prevent them from actually participating in this resistance and becoming severely damaged as a result.

The 1994 Northridge earthquake provided several examples where this was a cause of failure. In a preliminary reconnaissance report of that earthquake (EERI, 1994) it was stated: "Of much significance is the observation that six of the seven partial collapses (in modern precast concrete parking structures) seem to have been precipitated by damage to the gravity load system. Possibly, the combination of large lateral deformation and vertical load caused crushing in poorly confined columns that were not detailed to be part of the lateral load resisting system." The report also noted that: "Punching shear failures were observed in some structures at slab-to-column connections such as at the Four Seasons building in Sherman Oaks. The primary lateral load resisting system was a perimeter ductile frame that performed quite well. However, the interior slab-column system was incapable of undergoing the same lateral deflections and experienced punching failures."

In response to a preponderance of evidence, SEAOC successfully submitted a change to the *Uniform Building Code* in 1994 to clarify and strengthen the existing requirements intended to require deformation compatibility. The statement in support of that code change included the following reasons: "Deformation compatibility requirements have largely been ignored by the design community. In the 1994 Northridge earthquake, deformation-induced damage to elements which were not part of the lateral-force-resisting system resulted in structural collapse. Damage to elements of the lateral-framing system, whose behavior was affected by adjoining rigid elements, was also observed. This has demonstrated a need for stronger and clearer requirements.

The proposed changes attempt to emphasize the need for specific design and detailing of elements not part of the lateral system to accommodate expected seismic deformation...."

Language introduced in the 1997 *Provisions* was largely based on SEAOC's successful 1995 change to the *Uniform Building Code.* Rather than implicitly relying on designers to assume appropriate levels of stiffness, the new language in Sec. 5.2.2.4.3 explicitly requires that the "stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used" for the design of *components* that are not part of the lateral-force-resisting system. This was intended to keep designers from neglecting the potentially adverse stiffening effects that such *components* can have on structures. This section also includes a requirement to address shears that can be induced in structural *components* that are not part of the lateral-force-resisting system since sudden shear failures have been catastrophic in past earthquakes.

The exception in Sec. 5.2.4.3 is intended to encourage the use of intermediate or special detailing in beams and columns that are not part of the lateral-force-resisting system. In return for better detailing, such beams and columns are permitted to be designed to resist moments and shears from unamplified deflections. This reflects observations and experimental evidence that well-detailed *components* can accommodate large drifts by responding inelastically without losing significant vertical load carrying capacity.

5.2.2.5 Seismic Design Category F: Sec. 5.2.2.5 covers Category F, which is restricted to essential facilities on sites located within a few kilometers of major active faults. Because of the necessity for reducing risk (particularly in terms of protecting life safety or maintaining function by minimizing damage to nonstructural building elements, contents, equipment, and utilities), the height limitations for Category F are reduced. Again, the limits--100 ft (30 m) and 160 ft (49 m)--are arbitrary and require further study. The developers of these requirements believe that, at present, it is advisable to establish these limits, but the importance of having more stringent requirements for detailing the seismic-force-resisting system as well as the nonstructural *components* of the building must be stressed. Such requirements are specified in Sec. 5.2.6 and Chapters 8 through 12.

5.2.3 Structure Configuration: The configuration of a structure can significantly affect its performance during a strong earthquake that produces the ground motion contemplated in the *Provisions.* Configuration can be divided into two aspects, plan configuration and vertical configuration. The *Provisions* were basically derived for buildings having regular configurations. Past earthquakes have repeatedly shown that buildings having irregular configurations suffer greater damage than buildings having regular configurations. This situation prevails even with good design and construction. There are several reasons for this poor behavior of irregular structures. In a regular structure, inelastic demands produced by strong ground shaking tend to be well distributed throughout the structure, resulting in a dispersion of energy dissipation and damage. However, in irregular structures, inelastic behavior can concentrate in the zone of irregularity. resulting in rapid failure of structural elements in these areas. In addition, some irregularities introduce unanticipated stresses into the structure which designers frequently overlook when detailing the structural system. Finally, the elastic analysis methods typically employed in the design of structures often can not predict the distribution of earthquake demands in an irregular structure very well, leading to inadequate design in the zones of irregularity. For these reasons, these requirements are designed to encourage that buildings be designed to have regular configurations and to prohibit gross irregularity in buildings located on sites close to major active faults, where very strong ground motion and extreme inelastic demands can be experienced.

5.2.3.2 Plan Irregularity: Sec. 5.2.3.2 indicates, by reference to Table 5.2.3.2, when a building must be designated as having a plan irregularity for the purposes of the *Provisions*. A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass or vertical seismic resisting elements. Torsional effects in earthquakes can occur even when the static centers of mass and resistance coincide. For example, ground motion waves acting with a skew with respect to the building axis can cause torsion. Cracking or yielding in a nonsymmetrical fashion also can cause torsion. These effects also can magnify the torsion due to eccentricity between the static centers. For this reason, buildings having an eccentricity between the static center of mass and the static center of resistance in excess of 10 percent of the building dimension perpendicular to the direction of the seismic force should be classified as irregular. The vertical resisting *components* may be arranged so that the static centers of mass and resistance are within the limitations given above and still be unsymmetrically arranged so that the prescribed torsional forces would be unequally distributed to the various *components*. In the 1997 *Provisions*, torsional irregularities were subdivided into two categories, with a category of extreme irregularity having been created. Extreme torsional irregularities are prohibited for structures located very close to major active faults and should be avoided, when possible, in all structures.

There is a second type of distribution of vertical resisting *components* that, while not being classified as irregular, does not perform well in strong earthquakes. This arrangement is termed a core-type building with the vertical *components* of the seismic-force-resisting system concentrated near the center of the building. Better performance has been observed when the vertical *components* are distributed near the perimeter of the building. In recognition of the problems leading to torsional instability, a torsional amplification factor is introduced in Sec. 5.3.5.2.

A building having a regular configuration can be square, rectangular, or circular. A square or rectangular building with minor re-entrant corners would still be considered regular but large re-entrant corners creating a crucifix form would be classified as an irregular configuration. The response of the wings of this type of building is generally different from the response of the building as a whole, and this produces higher local forces than would be determined by application of the *Provisions* without modification. Other plan configurations such as H-shapes that have a geometrical symmetry also would be classified as irregular because of the response of the wings.

Significant differences in stiffness between portions of a diaphragm at a level are classified as irregularities since they may cause a change in the distribution of seismic forces to the vertical *components* and create torsional forces not accounted for in the normal distribution considered for a regular building. Examples of plan irregularities are illustrated in Figure C5.2.3.2.

Where there are discontinuities in the lateral force resistance path, the structure can no longer be considered to be "regular." The most critical of the discontinuities to be considered is the out-of-plane offset of vertical elements of the seismic force resisting elements. Such offsets impose vertical and lateral load effects on horizontal elements that are, at the least, difficult to provide for adequately.

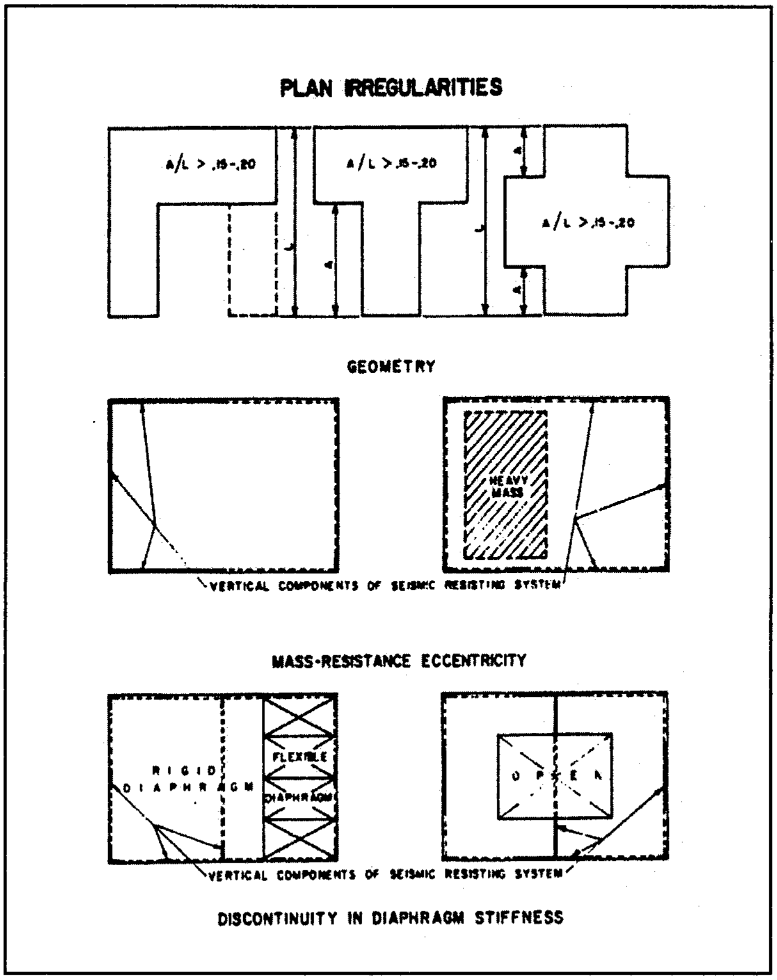


FIGURE C5.2.3.2 Building plan irregularities.

Where vertical elements of the lateral-force-resisting system are not parallel to or symmetric with major orthogonal axes, the static lateral force procedures of the *Provisions* cannot be applied as given and, thus, the structure must be considered to be "irregular."

5.2.3.3 Vertical Irregularity: Sec. 5.2.3.3 indicates, by reference to Table 5.2.3.3, when a structure must be considered to have a vertical irregularity. Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels that are significantly different from the distribution assumed in the equivalent lateral force procedure given in Sec. 5.3.

A moment resisting frame building might be classified as having a vertical irregularity if one story were much taller than the adjoining stories and the resulting decrease in stiffness that would normally occur was not, or could not be, compensated for. Examples of vertical irregularities are illustrated in Figure C5.2.3.3.

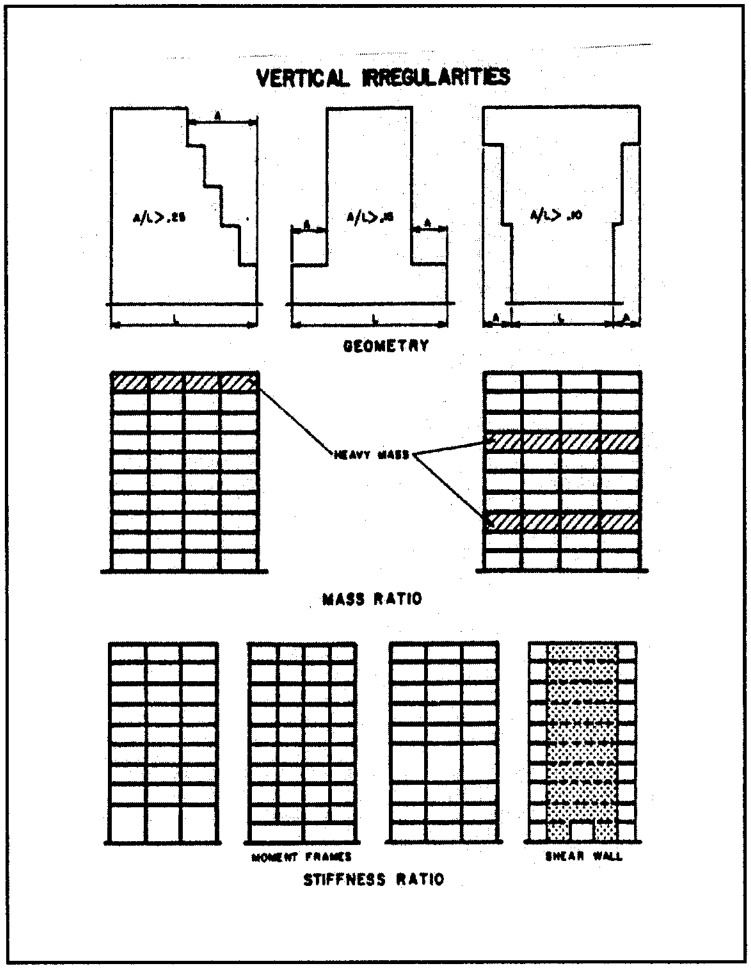


FIGURE C5.2.3.3 Building elevation irregularities.

A building would be classified as irregular if the ratio of mass to stiffness in adjoining stories differs significantly. This might occur when a heavy mass, such as a swimming pool, is placed at one level. Note that the exception in the *Provisions* provides a comparative stiffness ratio between stories to exempt structures from being designated as having a vertical irregularity of the types specified.

One type of vertical irregularity is created by unsymmetrical geometry with respect to the vertical axis of the building. The building may have a geometry that is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offsets in the vertical elements of the lateral-force-resisting system at one or more levels. An offset is considered to be significant if the ratio of the larger dimension to the smaller dimension is more than 130 percent. The building also would be considered irregular if the smaller dimension were below the larger dimension, thereby creating an inverted pyramid effect.

Weak story irregularities occur whenever the strength of a story to resist lateral demands is significantly less than that of the story above. This is because buildings with this configuration tend to develop all of their inelastic behavior at the weak story. This can result in a significant change in the deformation pattern of the building, with most earthquake induced displacement occurring within the weak story. This can result in extensive damage within the weak story and even instability and collapse. Note that an exception has been provided in Sec. 5.2.6.2.4 when there is considerable overstrength of the "weak" story.

In the 1997 *Provisions*, the soft story irregularity was subdivided into two categories with an extreme soft story category being created. Like weak stories, soft stories can lead to instability and collapse. Buildings with extreme soft stories are now prohibited on sites located very close to major active faults.

5.2.4 Redundancy: The 1997 *Provisions* introduced specific requirements intended to quantify the importance of redundancy. Many parts of the *Provisions*, particularly the response modification coefficients, *R*, were originally developed assuming that structures possess varying levels of redundancy that heretofore were undefined. *Commentary* Sec. 5.2.1 recommends that lower R values be used for non-redundant systems, but does not provide guidance on how to select and justify appropriate reductions. As a result, many non-redundant structures have been designed in the past using values of *R* that were intended for use in designing structures with higher levels of redundancy. For example, current *R* values for special moment resisting frames were initially established in the 1970s based on the then widespread use of complete or nearly complete frame systems in which all beam-column connections were designed to participate in the lateral-force-resisting system. High *R* values were justified by the large number of potential hinges that could form in such redundant systems, and the beneficial effects of progressive yield hinge formation described in Sec. C5.2.1. However, in recent years, economic pressures have encouraged the now prevalent use of much less redundant special moment frames with relatively few bays of moment resisting framing supporting large floor and roof areas. Similar observations have been made of other types of construction as well. Modern concrete and masonry shear wall buildings, for example, have many fewer walls than were once commonly provided in such buildings.

In order to quantify the effects of redundancy, the 1997 *Provisions* introduced the concept of a reliability factor, ρ, that is applied to the design earthquake loads in the basic load combination equations of Sec. 5.2.7, for structures in Seismic Design Categories D, E, and F. The value of the reliability factor ρ varies from 1 to 1.5. In effect this reduces the *R* values for less redundant structures and should provide greater economic incentive for the design of structures with well distributed lateral-force-resisting systems. The formulation for the equation from which ρ is derived is similar to that developed by SEAOC for inclusion in the 1997 edition of the *Uniform Building Code*. It bases the value of ρ on the floor area of the building and the parameter "r" which relates to the amount of the building's design lateral force carried by any single element.

There are many other considerations than just floor area and element/story shear ratios that should be considered in quantifying redundancy. Conceptually, the element demand/capacity ratios, types of mechanisms which may form, the individual characteristics of building systems and materials, building height, number of stories, irregularity, torsional resistance, chord and collector length, diaphragm spans, the number of lines of resistance, and the number of elements per line are all important and will intrinsically influence the level of redundancy in systems and their reliability.

The SEAOC proposed code change to the *1997 UBC* recommends addressing redundancy in irregular buildings by evaluating the ratio of element shear to design story shear, "r" only in the lower one-third height. However, many failures of buildings have occurred at and above mid-heights. Therefore, the *Provisions* base the ρ factor on the worst "r" for the least redundant story, which should then be applied throughout the height of the building.

The Applied Technology Council, in its ATC 19 report suggests that future redundancy factors be based on reliability theory. For example, if the number of hinges in a moment frame required to achieve a minimally redundant system were established, a redundancy factor for less redundant systems could be based on the relationship of the number of hinges actually provided to those required for minimally redundant systems. ATC suggests that similar relationships could be developed for shear wall systems using reliability theory. However, much work yet remains to be completed before such approaches will be ready for adoption into the *Provisions*.

The *Provisions* limit special moment resisting frames to configurations that provide maximum ρ values of 1.25 and 1.1, respectively, in Seismic Design Categories D, and E or F, to compensate for the strength based factor in what are typically drift controlled systems. Other seismic-force-resisting systems that are not typically drift controlled may be proportioned to exceed the maximum ρ factor of 1.5; however, it is not recommended that this be done.

5.2.5 Structural Analysis: Many of the standard procedures for the analysis of forces and deformations in structures subjected to earthquake ground motion are listed below in order of increasing rigor and expected accuracy:

1. Equivalent lateral force procedure (Sec. 5.4).
2. Modal analysis procedure (response spectrum analysis) (Sec. 5.5).
3. Linear response history analysis (Sec. 5.6).
4. Inelastic static procedure, involving incremental application of a pattern of lateral forces and adjustment of the structural model to account for progressive yielding under load application (push-over analysis) (Appendix 5).
5. Inelastic response history analysis involving step-by-step integration of the coupled equations of motion (Sec. 5.7).

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as presented in Sec. 5.8 or through a more complete analysis of this interaction as appropriate. Every procedure improves in rigor if combined with use of results from experimental research (not described in these *Provisions*).

The equivalent lateral force (ELF) procedure specified in Sec. 5.4 is similar in its basic concept to SEAOC recommendations in 1968, 1973, and 1974, but several improved features have been incorporated. A significant revision to this procedure, that more closely adopts the direct consideration of ground motion response spectra, was adopted in the 1997 *Provisions* in parallel with a similar concept developed by SEAOC.

The modal superposition method is a general procedure for linear analysis of the dynamic response of structures. In various forms, modal analysis has been widely used in the earthquake-resistant design of special structures such as very tall buildings, offshore drilling platforms, dams, and nuclear power plants, for a number of years; however, it use is also becoming more common for ordinary structures as well. Prior to the 1997 edition of the *Provisions*, the modal analysis procedure specified in Sec. 5.5 was simplified from the general case by restricting consideration to lateral motion in a single plane. Only one degree of freedom was required per floor for this type of analysis. In recent years, with the advent of high speed, desktop computers, and the proliferation of relatively inexpensive, user-friendly structural analysis software capable of performing three dimensional modal analyses, such simplifications have become unnecessary. Consequently, the 1997 *Provisions* adopted the more general approach describing a three-dimensional modal analysis of the structure. When modal analysis is specified by the *Provisions*, a three-dimensional analysis generally is required except in the case of highly regular structures or structures with flexible diaphragms.

The ELF procedure of Sec. 5.4 and the modal analysis procedure of Sec. 5.5 are both based on the approximation that the effects of yielding can be adequately accounted for by linear analysis of the seismic-force-resisting system for the design spectrum, which is the elastic acceleration response spectrum reduced by the response modification factor, R. The effects of the horizontal component of ground motion perpendicular to the direction under consideration in the analysis, the vertical component of ground motion, and torsional motions of the structure are all considered in the same simplified approaches in the two procedures. The main difference between the two procedures lies in the distribution of the seismic lateral forces over the building. In the modal analysis procedure, the distribution is based on properties of the natural vibration modes, which are determined from the mass and stiffness distribution. In the ELF procedure, the distribution is based on simplified formulas that are appropriate for regular structures as specified in Sec. 5.4.3. Otherwise, the two procedures are subject to the same limitations.

The simplifications inherent in the ELF procedure result in approximations that are likely to be inadequate if the lateral motions in two orthogonal directions and the torsional motion are strongly coupled. Such would be the case if the building were irregular in its plan configuration (see Sec. 5.2.3.2) or if it had a regular plan but its lower natural frequencies were nearly equal and the centers of mass and resistance were nearly coincident. The modal analysis method introduced in the 1997 *Provisions* includes a general model that is more appropriate for the analysis of such structures. It requires at least three degrees of freedom per floor--two translational and one torsional motion.

The methods of modal analysis can be generalized further to model the effect of diaphragm flexibility, soil-structure interaction, etc. In the most general form, the idealization would take the form of a large number of mass points, each with six degrees of freedom (three translation and three rotational) connected by generalized stiffness elements.

The ELF procedure (Sec. 5.4) and the modal analysis procedure are all likely to err systematically on the unsafe side if story strengths are distributed irregularly over height. This feature is likely to lead to concentration of ductility demand in a few stories of the building. The inelastic static (or so-called pushover) procedure is a method to more accurately account for irregular strength distribution. However, it also has limitations and is not particularly applicable to tall structures or structures with relatively long fundamental periods of vibration.

The actual strength properties of the various *components* of a structure can be explicitly considered only by a nonlinear analysis of dynamic response by direct integration of the coupled equations of motion. This method has been used extensively in earthquake research studies of inelastic structural response. If the two lateral motions and the torsional motion are expected to be essentially uncoupled, it would be sufficient to include only one degree of freedom per floor, the motion in the direction along which the structure is being analyzed; otherwise at least three degrees of freedom per floor, two translational motions and one torsional, should be included. It should be recognized that the results of a nonlinear response history analysis of such mathematical structural models are only as good as are the models chosen to represent the structure vibrating at amplitudes of motion large enough to cause significant yielding during strong ground motions. Furthermore, reliable results can be achieved only by calculating the response to several ground motions--recorded accelerograms and/or simulated motions--and examining the statistics of response.

It is possible with presently available computer programs to perform two- and three-dimensional inelastic analyses of reasonably simple structures. The intent of such analyses could be to estimate the sequence in which *components* become inelastic and to indicate those *components* requiring strength adjustments so as to remain within the required ductility limits. It should be emphasized that with the present state of the art in analysis, there is no one method that can be applied to all types of structures. Further, the reliability of the analytical results are sensitive to:

1. The number and appropriateness of the input motion records,
2. The practical limitations of mathematical modeling including interacting effects of inelastic elements,
3. The nonlinear solution algorithms, and
4. The assumed member hysteretic behavior.

Because of these sensitivities and limitations, the maximum base shear produced in an inelastic analysis should not be less than that required by Sec. 5.4.

The least rigorous analytical procedure that may be used in determining the design seismic forces and deformations in structures depends on the Seismic Design Category and the structural characteristics (in particular, regularity). Regularity is discussed in Sec. 5.2.3.

Neither regular nor irregular buildings in Seismic Design Category A are required to be analyzed as a whole for seismic forces, but certain minimum requirements are given in Sec. 5.2.5.1. In addition, there is a requirement that Seismic Design Category A structure should be evaluated for a total lateral force equal to a nominal percentage of their effective weight. The purpose of this provision is to assure that a complete lateral-force-resisting system is provided for all structures. Although this requirement was first introduced in the 1997 edition of the *Provisions*, in the 2000 edition it was formalized and termed the Index force Procedure (Sec. 5.3).

For the higher Seismic Design Categories, the ELF procedure is the minimum level of analysis except that a more rigorous procedure is required for some Category D, E and F structures as identified in Table 5.2.5.1. The modal analysis procedure adequately addresses vertical irregularities of stiffness, mass, or geometry, as limited by the *Provisions*. Other irregularities must be carefully considered.

The basis for the ELF procedure and its limitations were discussed above. It is adequate for most regular structures; however, the designer may wish to employ a more rigorous procedure (see list of procedures at beginning of this section for those regular *structures* where it may be inadequate). The ELF procedure is likely to be inadequate in the following cases:

1. Structures with irregular mass and stiffness properties in which case the simple equations for vertical distribution of lateral forces (Eq. 5.3.4-1 and 5.3.4-2) may lead to erroneous results;
2. Structures (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strongly coupled; and
3. Structures with irregular distribution of story strengths leading to possible concentration of ductility demand in a few stories of the building.

In such cases, a more rigorous procedure that considers the dynamic behavior of the structure should be employed.

Structures with certain types of vertical irregularities may be analyzed as regular *structures* in accordance with the requirements of Sec. 5.4. These structures are generally referred to as setback structures. The following procedure may be used:

1. The base and tower portions of a building having a setback vertical configuration may be analyzed as indicated in (2) below if:
   a. The base portion and the tower portion, considered as separate *structures* , can be classified as regular and
   b. The stiffness of the top story of the base is at least five times that of the first story of the tower.

   When these conditions are not met, the building must be analyzed in accordance with Sec. 5.4.
2. The base and tower portions may be analyzed as separate *structures* in accordance with the following:
   a. The tower may be analyzed in accordance with the procedures in Sec. 5.3 with the base taken at the top of the base portion.
   b. The base portion then must be analyzed in accordance with the procedures in Sec. 5.3 using the height of the base portion of h_n and with the gravity load and seismic base shear forces of the tower portion acting at the top level of the base portion.

The design requirements in Sec. 5.5 include a simplified version of modal analysis that accounts for irregularity in mass and stiffness distribution over the height of the building. It would be adequate, in general, to use the ELF procedure for *structures* whose floor masses and cross-sectional areas and moments of inertia of structural members do not differ by more than 30 percent in adjacent floors and in adjacent stories.

For other structures, the following procedure should be used to determine whether the modal analysis procedures of Sec. 5.5 should be used:

1. Compute the story shears using the ELF procedure specified in Sec. 5.4.
2. On this basis, approximately dimension the structural members, and then compute the lateral displacements of the floor.
3. Replace h in Eq. 5.4.3-2 with these displacements, and recompute the lateral forces to obtain the revised story shears.

4. If at any story the recomputed story shear differs from the corresponding value as obtained from the procedures of Sec. 5.4 by more than 30 percent, the building should be analyzed using the procedure of Sec. 5.5. If the difference is less than this value, the building may be designed using the story shear obtained in the application of the present criterion and the procedures of Sec. 5.5 are not required.

Application of this procedure to these structures requires far less computational effort than the use of the modal analysis procedure of Sec. 5.5. In the majority of the *structures*, use of this procedure will determine that modal analysis need not be used and will also furnish a set of story shears that practically always lie much closer to the results of modal analysis than the results of the ELF procedure.

This procedure is equivalent to a single cycle of Newmark's method for calculation of the fundamental mode of vibration. It will detect both unusual shapes of the fundamental mode and excessively high influence of higher modes. Numerical studies have demonstrated that this procedure for determining whether modal analysis must be used will, in general, detect cases that truly should be analyzed dynamically; however, it generally will not indicate the need for dynamic analysis when such an analysis would not greatly improve accuracy.

5.2.5.2. Application of Loading: Earthquake forces act in both principal directions of the building simultaneously, but the earthquake effects in the two principal directions are unlikely to reach their maximum simultaneously. This section provides a reasonable and adequate method for combining them. It requires that structural elements be designed for 100 percent of the effects of seismic forces in one principal direction combined with 30 percent of the effects of seismic forces in the orthogonal direction.

The following combinations of effects of gravity loads, effects of seismic forces in the x-direction, and effects of seismic forces in the y-direction (orthogonal to x-direction) thus pertain:

gravity ± 100% of x-direction ± 30% of y-direction
gravity ± 30% of x-direction ± 100% of y-direction

The combination and signs (plus or minus) requiring the greater member strength are used for each member. Orthogonal effects are slight on beams, girders, slabs, and other horizontal elements that are essentially one-directional in their behavior, but they may be significant in columns or other vertical members that participate in resisting earthquake forces in both principal directions of the building. For two-way slabs, orthogonal effects at slab-to-column connections can be neglected provided the moment transferred in the minor direction does not exceed 30 percent of that transferred in the orthogonal direction and there is adequate reinforcement within lines one and one-half times the slab thickness either side of the column to transfer all the minor direction moment.

5.2.6 Design and Detailing Requirements: The design and detailing requirements for *components* of the seismic-force-resisting system are stated in this section. The combination of load effects is specified in Sec. 5.2.7. The requirements of this section are spelled out in considerable detail. The major reasons for this are presented below.

The provision of detailed design ground motions and requirements for analysis of the structure do not by themselves make a building earthquake resistant. Additional design requirements are necessary to provide a consistent degree of earthquake resistance in buildings. The more severe the expected seismic ground motions, the more stringent these additional design requirements should be. Not all of the necessary design requirements are expressed in codes, and although experienced seismic design engineers account for them, engineers lacking experience in the design and construction of earthquake-resistant structures often overlook them. Considerable uncertainties exist regarding:

1. The actual dynamic characteristics of future earthquake motions expected at a building site;
2. The soil-structure-foundation interaction;
3. The actual response of buildings when subjected to seismic motions at their foundations; and
4. The mechanical characteristics of the different structural materials, particularly when they undergo significant cyclic straining in the inelastic range that can lead to severe reversals of strains.

It should be noted that the overall inelastic response of a structure is very sensitive to the inelastic behavior of its critical regions, and this behavior is influenced, in turn, by the detailing of these regions.

Although it is possible to counteract the consequences of these uncertainties by increasing the level of design forces, it is considered more feasible to provide a building system with the largest energy dissipation consistent with the maximum tolerable deformations of nonstructural *components* and equipment. This energy dissipation capacity, which is usually denoted simplistically as "ductility," is extremely sensitive to the detailing. Therefore, in order to achieve such a large energy dissipation capacity, it is essential that stringent design requirements be used for detailing the structural as well as the nonstructural *components* and their connections or separations. Furthermore, it is necessary to have good quality control of materials and competent inspection. The importance of these factors has been clearly demonstrated by the building damage observed after both moderate and severe earthquakes.

It should be kept in mind that a building's response to seismic ground motion most often does not reflect the designer's or analyst's original conception or modeling of the structure on paper. What is reflected is the manner in which the building was constructed in the field. These requirements emphasize the importance of detailing and recognize that the detailing requirements should be related to the expected earthquake intensities and the importance of the building's function and/or the density and type of occupancy. The greater the expected intensity of earthquake ground-shaking and the more important the building function or the greater the number of occupants in the building, the more stringent the design and detailing requirements should be. In defining these requirements, the *Provisions* uses the concept of Seismic Design Categories (Tables 4.2.1a and 4.2.1b), which relate to the design ground motion severities, given by the spectral response acceleration coefficients S_{DS} and S_{D1} (Sec. 4.1.1) and the Seismic Use Group (Sec. 1.3).

5.2.6.1 Seismic Design Category A: Because of the very low seismicity associated with sites with S_{DS} less than 0.25g and S_{D1} less than 0.10g , it is considered appropriate for Category A buildings to require only a complete lateral-force-resisting system. good quality of construction materials and adequate ties and anchorage as specified in this section. Category A buildings will be constructed in a large portion of the United States that is generally subject to strong winds but low earthquake risk. Those promulgating construction regulations for these areas may wish to consider many of the low-level seismic requirements as being suitable to reduce the windstorm risk. Since the *Provisions* considers only earthquakes, no other requirements are prescribed for Category A buildings. Only a complete lateral-force-resisting system, ties, and wall anchorage are required by these *Provisions*.

5.2.6.1.1 Connections: The analysis of a structure and the provision of a design ground motion alone do not make a structure earthquake resistant; additional design requirements are necessary to provide adequate earthquake resistance in buildings. Experienced seismic designers normally fill these requirements, but because some were not formally specified, they often are overlooked by inexperienced engineers.

Probably the most important single attribute of an earthquake-resistant building is that it is tied together to act as a unit. This attribute not only is important in earthquake-resistant design, but also is indispensable in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. Sec. 5.2.6.1.1 requires that all parts of the building (or unit if there are separation joints) be so tied together that any part of the structure is tied to the rest to resist a force of $S_{DS}/7.5$ (with a minimum of 5 percent g) times the weight of the smaller. In addition, beams must be tied to their supports or columns and columns to footings for a minimum of 5 percent of the dead and live load reaction.

Certain connections of buildings with plan irregularities must be designed for higher forces than calculated due to the simplifying assumptions used in the analysis by Sec. 5.3, 5.4, and 5.5 (see Sec. 5.2.6.4.2).

5.2.6.1.2 Anchorage of Concrete or Masonry Walls: One of the major hazards from buildings during an earthquake is the pulling away of heavy masonry or concrete walls from floors or roofs. Although requirements for the anchorage to prevent this separation are common in highly seismic areas, they have been minimal or nonexistent in most other parts of the country. This section requires that anchorage be provided in any locality to the extent of $400S_{DS}$ pounds per linear foot (plf) or 5,840 times S_{DS} Newtons per meter (N/m). This requirement alone may not provide complete earth-quake-resistant design, but observations of earthquake damage indicate that it can greatly increase the earthquake resistance of buildings and reduce hazards in those localities where earthquakes may occur but are rarely damaging.

5.2.6.2 Seismic Design Category B: Category B and Category C buildings will be constructed in the largest portion of the United States. Earthquake-resistant requirements are increased appreciably over Category A requirements, but they still are quite simple compared to present requirements in areas of high seismicity.

The Category B requirements specifically recognize the need to design diaphragms, provide collector bars, and provide reinforcing around openings. There requirements may seem elementary and obvious but, because they are not specifically covered in many codes, some engineers totally neglect them.

5.2.6.2.4 Nonredundant Systems: Design consideration should be given to potentially adverse effects where there is a lack of redundancy. Because of the many unknowns and uncertainties in the magnitude and characteristics of earthquake loading, in the materials and systems of construction for resisting earthquake loadings and in the methods of analysis, good earthquake engineering practice has been to provide as much redundancy as possible in the seismic-force-resisting system of buildings.

Redundancy plays an important role in determining the ability of the building to resist earthquake forces. In a structural system without redundant *components*, every component must remain operative to preserve the integrity of the building structure. On the other hand, in a highly redundant system, one or more redundant *components* may fail and still leave a structural system that retains its integrity and can continue to resist lateral forces, albeit with diminished effectiveness.

Redundancy often is accomplished by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic-force-resisting system. These multiple points of resistance can prevent a catastrophic collapse due to distress or failure of a member or joint. (The overstrength characteristics of this type of frame were discussed in the commentary on Sec. 5.2.1.)

The designer should be particularly aware of the proper selection of R when using only one or two one-bay rigid frames in one direction for resisting seismic loads. A single one-bay frame or a pair of such frames provides little redundancy so the designer may wish to consider a modified (smaller) R to account for a lack of redundancy. As more one-bay frames are added to the system, however, overall system redundancy increases. The increase in redundancy is a function of frame placement and total number of frames.

Redundant characteristics also can be obtained by providing several different types of seismic-force-resisting systems in a building. The backup system can prevent catastrophic effects if distress occurs in the primary system.

In summary, it is good practice to incorporate redundancy into the seismic-force-resisting system and not to rely on any system wherein distress in any member may cause progressive or catastrophic collapse.

5.2.6.2.5 Collector Elements: Many buildings have shear walls or other bracing elements that are not uniformly spaced around the diaphragms. Such conditions require that collector or drag members be provided. A simple illustration is shown in Figure C5.2.6.2.5.

Consider a building as shown in the plan with four short shear walls at the corners arranged as shown. For north-south earthquake forces, the diaphragm shears on Line AB are uniformly distributed between A and B if the chord reinforcing is assumed to act on Lines BC and AD. However, wall A is quite short so reinforcing steel is required to collect these shears and transfer them to the wall. If Wall A is a quarter of the length of AB, the steel must carry, as a minimum, three-fourths of the total shear on Line AB. The same principle is true for the other walls. In Figure C5.2.6.2.5 reinforcing is required to collect the shears or drag the forces from the diaphragm into the shear wall. Similar collector elements are needed in most shear walls and some frames.

5.2.6.2.6 Diaphragms: Diaphragms are deep beams or trusses that distribute the lateral loads from their origin to the *components* where they are resisted. As such, they are subject to shears, bending moments, direct stresses (truss member, collector elements), and deformations. The deformations must be minimized in some cases because they could overstress the walls to which they are connected. The amount of deflection permitted in the diaphragm must be related to the ability of the walls (normal to the direction being analyzed) to deflect without failure.

A detail commonly overlooked by many engineers is the requirement to tie the diaphragm together so that it acts as a unit. Wall anchorages tend to tear off the edges of the diaphragm; thus, the ties must be extended into the diaphragm so as to develop adequate anchorage. During the San Fernando earthquake, seismic forces from the walls caused separations in roof diaphragms 20 or more ft (6 m) from the edge in several industrial buildings.

When openings occur in shear walls, diaphragms, etc., it is not adequate to only provide temperature trim bars. The chord stresses must be provided for and the chords anchored to develop the chord stresses by embedment. The embedment must be sufficient to take the reactions without overstressing the material in any respect. Since the design basis depends on an elastic analysis, the internal force system should be compatible with both static and the elastic deformations.

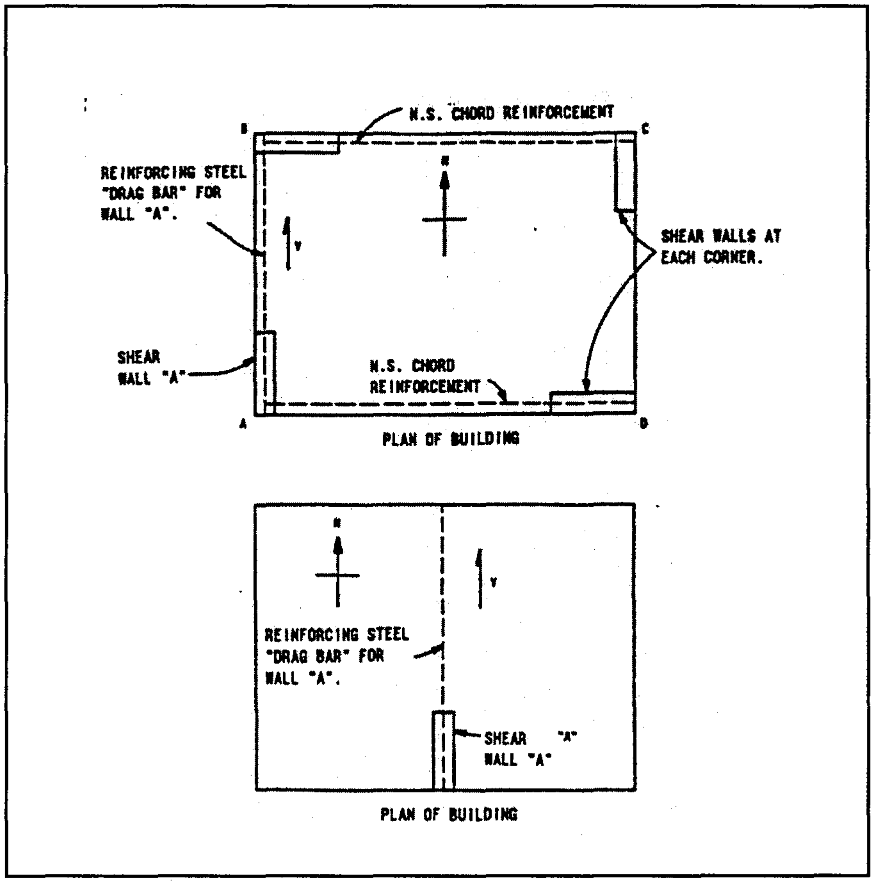


FIGURE C5.2.6.2.5 Collector element used to (a) transfer shears and (b) transfer drag forces from diaphragm to shear wall.

5.2.6.2.7 Bearing Walls: A minimum anchorage of bearing walls to diaphragms or other resisting elements is specified. To ensure that the walls and supporting framing system interact properly, it is required that the interconnection of dependent wall elements and connections to the framing system have sufficient ductility or rotational capacity, or strength, to stay as a unit. Large shrinkage or settlement cracks can significantly affect the desired interaction.

5.2.6.2.8 Inverted Pendulum-Type Structures: Inverted pendulum-type structures have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. Often the structures are T-shaped with a single column supporting a beam or slab at the top. For such a structure, the lateral motion is accompanied by rotation of the horizontal element of the *T* due to rotation at the top of the column, resulting in vertical accelerations acting in opposite directions on the overhangs of the structure. Dynamic response amplifies this rotation; hence, a bending moment would be induced at the top of the column even though the procedures of

Sec. 5.4.1 and 5.4.4 would not so indicate. A simple provision to compensate for this is specified in this section. The bending moments due to the lateral force are first calculated for the base of the column according to the requirements of Sec. 5.4.1 and 5.4.4. One-half of the calculated bending moment at the base is applied at the top and the moments along the column are varied from 1.5 M at the base to 0.5 M at the top. The addition of one-half the moment calculated at the base in accordance with Sec. 5.4.1 and 5.4.4 is based on analyses of inverted pendulums covering a wide range of practical conditions.

5.2.6.2.9 Anchorage of Nonstructural Systems: Anchorage of nonstructural systems and *components* of buildings is required when prescribed in Chapter 6.

5.2.6.3 Seismic Design Category C: The material requirements in Chapters 8 through 12 for Category C are somewhat more restrictive than those for Categories A and B. Also, a nominal interconnection between pile caps and caissons is required.

5.2.6.4 Seismic Design Category D: Category D requirements compare roughly to present design practice in California seismic areas for buildings other than schools and hospitals. All moment resisting frames of concrete or steel must meet ductility requirements. Interaction effects between structural and nonstructural elements must be investigated. Foundation interaction requirements are increased.

5.2.7 Combination of Load Effects: The load combination statements in the *Provisions* combine the effects of structural response to horizontal and vertical ground accelerations. They do not show how to combine the effect of earthquake loading with the effects of other loads. For those combinations, the user is referred to ASCE 7. The pertinent combinations are:

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(Additive)}$$
$$0.9D + 1.0E \qquad \text{(Counteracting)}$$

where D, E, L, and S are, respectively, the dead, earthquake, live, and snow loads.

The design basis expressed in Sec. 5.2.1 reflects the fact that the specified earthquake loads are at the design level without amplification by load factors; thus, for sufficiently redundant structures, a load factor of 1.0 is assigned to the earthquake load effects in Eq. 5.2.7-1 and 5.2.7-2.

In Eq. 5.2.7-1 and 5.2.7-2 , a factor of $0.2S_{DS}$ was placed on the dead load to account for the effects of vertical acceleration. The $0.2S_{DS}$ factor on dead load is not intended to represent the total vertical response. The concurrent maximum response of vertical accelerations and horizontal accelerations, direct and orthogonal, is unlikely and, therefore, the direct addition of responses was not considered appropriate.

The ρ factor was introduced into Eq. 5.2.7-1 and 5.2.7-2 in the 1997 *Provisions*. This factor, determined in accordance with Sec. 5.2.4, relates to the redundancy inherent in the lateral-force-resisting system and is, in essence, a reliability factor, penalizing designs which are likely to be unreliable due to concentration of the structure's resistance to lateral forces in a relatively few elements.

There is very little research that speaks directly to the merits of redundancy in buildings for seismic resistance. The SAC joint venture recently studied the relationships between damage to welded steel moment frame connections and redundancy (Bonowitz, et al, 1995). While this study found no specific correlation between damage and the number of bays of moment resisting framing per moment frame, it did find increased rates of damage in connections that resisted larger floor areas. This study included modern low-, mid- and high-rise steel buildings.

Another study (Wood, 1991) that addresses the potential effects of redundancy evaluated the performance of 165 Chilean concrete buildings ranging from 6 to 23 stories in height. These concrete shear wall buildings with non-ductile details and no boundary elements experienced moderately strong shaking (MMI VII to VIII) with a strong shaking duration of over 60 seconds, yet performed well. One plausible explanation for this generally good performance was the substantial amount of wall area (2 to 4 percent of the floor area) commonly used in Chile. However, Wood's study found no correlation between damage rates and higher redundancy in buildings with wall areas greater than 2 percent.

The special load combination of Sec. 5.2.7.1 is intended to address those situations where failure of an isolated, individual, brittle element can result in the loss of a complete lateral-force-resisting system or in instability and collapse. This section has evolved over several editions. In the 1991 Edition, a $2R/5$ factor was introduced to better represent the behavior of elements sensitive to overstrength in the remainder of the seismic resisting system or in specific other structural *components*. The particular number was selected to correlate with the $3R_w/8$ factor that had been introduced in Structural Engineers Association of California (SEAOC) recommendations and the *Uniform Building Code*. This is a somewhat arbitrary factor that attempts to quantify the maximum force that can be delivered to sensitive elements based on historic observation that the real force that could develop in a structure may be 3 to 4 times the design levels. In the 1997 *Provisions*, an attempt was made to determine this force more rationally through the assignment of the Ω_0 factor in Table 5.2.2, dependent on the individual system.

The special load combinations of Eq. 5.2.7.1-1 and 5.2.7.1-2 were first introduced in the 1991 Edition of the *Provisions*, for the design of elements that could fail in an undesirable manner when subjected to demands that are significantly larger than those used to proportion them. It recognizes the fact that the actual response (forces and deformations) developed by a structure subjected to the design earthquake ground motion will be substantially larger than that predicted by the design forces. Through the use of the Ω_o coefficient, this special equation provides an estimate of the maximum forces actually likely to be experienced by an element.

When originally introduced in the 1991 *Provisions*, the overstrength factor Ω_o was represented by the factor $2R/5$. That particular value was selected to correlate with the $3R_w/8$ factor that had been previously introduced in Structural Engineers Association of California (SEAOC) recommendations and the *Uniform Building Code* in 1988. Typically, both of these factors resulted in a three to four fold amplification in the design force levels, based on the historic judgment that the real forces experienced by a structure in a major earthquake are probably on the order of 3 to 4 times the design force levels.

In recent years, a number of researchers have investigated the factors that permit structures designed for reduced forces to survive design earthquakes. Although these studies have principally been focused on the development of more reliable response modification coefficients, R, they have identified the importance of structural overstrength, and identified a number of sources of such overstrength. This has made it possible to replace the single $2R/5$ factor formerly contained in the *Provisions* with a more system-specific estimate, represented by the Ω_o coefficient.

It is recognized, that no single value, whether obtained by formula related to the R factor or otherwise obtained will provide a completely accurate estimate for the overstrength of all structures with a given seismic-force-resisting system. However, most structures designed with a given lateral-force-resisting system, will fall within a range of overstrength values. Since the purpose of the Ω_0 factor in Eq. 5.2.7.1-1 and 5.2.7.1-2 is to estimate the maximum force that can be delivered to a component that is sensitive to overstress, the values of this factor tabulated in Table 5.2.2 are intended to be representative of the larger values in this range for each system.

Figure C5.2.7 and the following discussion explore some of the factors that contribute to structural overstrength. The figure shows a plot of lateral structural strength vs. displacement for an elastic-perfectly-plastic structure. In addition, it shows a similar plot for a more representative real structure, that posses significantly more strength than the design strength. This real strength is represented by the lateral force F_n. Essentially, the Ω_0 coefficient is intended to be a somewhat conservative estimate of the ratio of F_n to the design strength F_E/R. As shown in the figure, there are three basic *components* to the overstrength. These are the design overstrength (Ω_D), the material overstrength (Ω_M) and the system overstrength (Ω_S). Each of these is discussed separately. The design overstrength (Ω_D) is the most difficult of the three to estimate. It is the difference between the lateral base shear force at which the first significant yield of the structure will occur (point 1 in the figure) and the minimum specified force given by F_E/R. To some extent, this is system dependent. Systems that are strength controlled, such as most braced frames and shear wall structures, will typically have a relatively low value of design overstrength, as most designers will seek to optimize their designs and provide a strength that is close to the minimum specified by the *Provisions*. For such structures, this portion of the overstrength coefficient could be as low as 1.0.

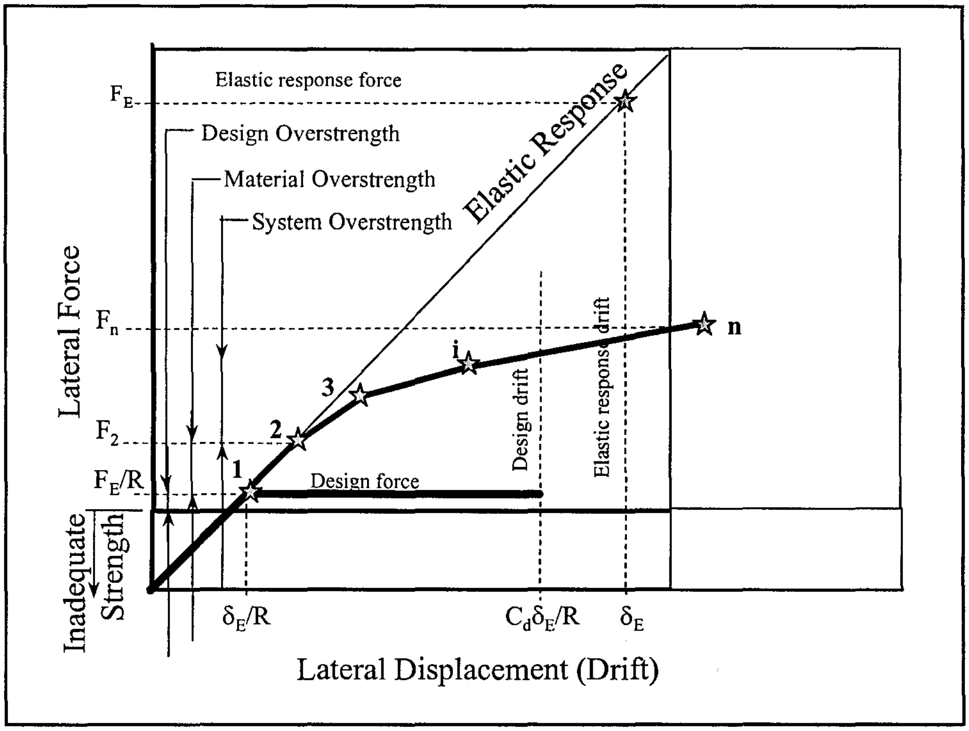


FIGURE C5.2.7 Factors affecting overstrength.

Drift controlled systems such as moment frames, however, will have substantially larger design overstrengths since it will be necessary to oversize the sections of such structures in order to keep the lateral drifts within prescribed limits. In a recent study of a number of special moment resisting steel frames conducted by the SAC Joint Venture design overstrengths on the order of a factor of two to three were found to exist (*Analytical Investigation of Buildings Affected by the 1994 Northridge Earthquake, Volumes 1 and 2*, SAC 95-04A and B. SAC Joint Venture, Sacramento, CA, 1995). Design overstrength is also potentially regionally dependent. The SAC study was conducted for frames in Seismic Design Category D and E, which represent the most severe design conditions. For structures in Seismic Design Categories A, B and C, seismic force resistance would play a less significant role in the sizing of frame elements to control drifts, and consequently, design overstrengths for these systems would be somewhat lower. It seems reasonable to assume that this portion of the design overstrength for special moment frame structures is on the order of 2.0.

Architectural design considerations have the potential to play a significant role in design overstrength. Some architectural designs will incorporate many more and larger lateral force resisting elements than are required to meet the strength and drift limitations of the code. An example of this are warehouse type structures, wherein the massive perimeter walls of the structure can provide very large lateral strength. However, even in such structures, there is typically some limiting element, such as the diaphragm, that prevents the design overstrength from becoming uncontrollably large. Thus, although the warehouse structure may have very large lateral resistance in its shear walls, typically the roof diaphragm will have a lateral force resisting capacity comparable to that specified as a minimum by the *Provisions*.

Finally, the structural designer can affect the design overstrength. While some designers seek to optimize their structures with regard to the limitations contained in the *Provisions*, others will seek to intentionally provide greater strength and drift control than required. Typically design overstrength intentionally introduced by the designer will be on the order of 10 percent of the minimum required strength, but it may range as high as 50 to 100 percent in some cases. A factor of 1.2 should probably be presumed for this portion of the design overstrength to include the effects of both architectural and structural design overstrength. Designers who intentionally provide greater design overstrength should keep in mind that the Ω_0 factors used in their designs should be adjusted accordingly.

Material overstrength (Ω_M) results from the fact that the design values used to proportion the elements of a structure are specified by the *Provisions* to be conservative lower bound estimates of the actual probable strengths of the structural materials and their effective strengths in the as-constructed structure. It is represented in the figure by the ratio of F_2/F_1, where F_2 and F_1 are respectively the lateral force at points 2 and 1 on the curve. All structural materials have considerable variation in the strengths that can be obtained in given samples of the material from a specific grade. The design requirements typically base proportioning requirements on minimum specified values that are further reduced through strength reduction (ϕ) factors. The actual expected strength of the as-constructed structure is significantly higher than this design value and should be calculated using the mean strength of the material, based on statistical data, by removal of the ϕ factor from the design equation, and by providing an allowance for strain hardening, where significant yielding is expected to occur. Code requirements for reinforced masonry, concrete and steel have historically used a factor of 1.25 to account for the ratio of mean to specified strength and the effects of strain hardening. Considering a typical capacity reduction factor on the order of 0.9, this would indicate that the material overstrength for systems constructed of these materials would be on the order of 1.25/0.9, or 1.4.

System overstrength (Ω_S) is the ratio of the ultimate lateral force the structure is capable of resisting, F_n in the figure, to the actual force at which first significant yield occurs, F_2 in the figure. It is dependent on the amount of redundancy contained in the structure as well as the extent to which the designer has optimized the various elements that participate in lateral force resistance. For structures, with a single lateral force resisting element, such as a braced frame structure with a single bay of bracing, the system overstrength (Ω_S) factor would be 1.0, since once the brace in the frame yields, the system becomes fully yielded. For structures that have a number of elements participating in lateral seismic force resistance, whether or not actually intended to do so, the system overstrength will be significantly larger than this, unless the designer has intentionally optimized the structure such that a complete side sway mechanism develops at the level of lateral drift at which the first actual yield occurs.

Structural optimization is most likely to occur in structures where the actual lateral force resistance is dominated by the design of elements intended to participate as part of the lateral-force-resisting system, and where the design of those elements is dominated by seismic loads, as opposed to gravity loads. This would include concentric braced frames and eccentric braced frames in all Seismic Design Categories and Special Moment Frames in Seismic Design Categories D and E. For such structures, the system overstrength may be taken on the order of 1.1. For dual system structures, the system overstrength is set by the *Provisions* at an approximate minimum value of 1.25. For structures where the number of elements that actually resist lateral forces is based on other than seismic design considerations, the system overstrength may be somewhat larger. In light framed residential construction, for example, the number of walls is controlled by architectural rather than seismic design consideration. Such structures may have a system overstrength on the order of 1.5. Moment frames, the design of which is dominated by gravity load considerations can easily have a system overstrength of 2.0 or more. This affect is somewhat balanced by the fact that such frames will have a lower design overstrength related to the requirement to increase section sizes to obtain drift control. Table C5.2.7-1 presents some possible ranges of values for the various *components* of overstrength for various structural systems as well as the overall range of values that may occur for typical structures.

TABLE C5.2.7-1 Typical Range of Overstrength for Various Systems

Structural System	Design Overstrength Ω_D	Material Overstrength Ω_M	System Overstrength Ω_S	Ω_0
Special Moment Frames Steel & Concrete	1.5-2.5	1.2-1.6	1.0-1.5	2-3.5
Intermediate Moment Frames Steel & Concrete	1.0-2.0	1.2-1.6	1.0-2.0	2-3.5
Ordinary Moment Frames Steel & Concrete	1.0-1.5	1.2-1.6	1.5-2.5	2-3.5
Masonry Wall Frames	1.0-2.0	1.2-1.6	1.0-1.5	2-2.5
Braced Frames	1.5-2.0	1.2-1.6	1.0-1.5	1.5-2
Reinforced Bearing Wall	1.0-1.5	1.2-1.6	1.0-1.5	1.5-2.5
Reinforced Infill Wall	1.0-1.5	1.2-1.6	1.0-1.5	1.5-2.5
Unreinforced Bearing Wall	1.0-2.0	0.8-2.0	1.0-2.0	2-3
Unreinforced Infill Wall	1.0-2.0	0.8-2.0	1.0-2.0	2-3
Dual System Bracing & Frame	1.1-1.75	1.2-1.6	1.0-1.5	1.5-2.5
Light Bearing Wall Systems	1.0-0.5	1.2-2.0	1.0-2.0	2.5-3.5

In recognition of the fact that it is difficult to accurately estimate the amount of overstrength a structure will have, based solely on the type of seismic-force-resisting system that is present, in lieu of using the values of the overstrength coefficient Ω_0 provided in Table 5.2.2, designers are encouraged to base the maximum forces used in Eqs. 5.2.7.1-1and 5.2.7.1-2 on the results of a suitable nonlinear analysis of the structure. Such analyses should use the actual expected, rather than specified values, of material and section properties. Appropriate forms of such analyses could include a plastic mechanism analysis, a static pushover analysis or a nonlinear time history analysis. If a plastic mechanism analysis is utilized, the maximum seismic force that ever could be produced in the structure, regardless of the ground motion experienced is, estimated. If static pushover or nonlinear time history analyses are utilized, the forces utilized for design as the maximum force, should probably be that determined for Maximum Considered Earthquake level ground shaking demands.

While overstrength can be quite beneficial in permitting structures to resist actual seismic demands that are larger than those for which they have been specifically designed, it is not always beneficial. Some elements incorporated in structures behave in a brittle manner and can fail in an abrupt manner if substantially overloaded. The existence of structural overstrength results in a condition where such overloads are likely to occur, unless they are specifically accounted for in the design process. This is the purpose of Eq. 5.2.7.1-1 and 5.2.7.1-2.

One case where structural overstrength should specifically be considered is in the design of column elements beneath discontinuous braced frames and shear walls, such as occurs at vertical in-plane and out-of-plane irregularities. Overstrength in the braced frames and shear walls could cause buckling failure of such columns with resulting structural collapse. Columns subjected to tensile loading in which splices are made using partial penetration groove welds, a type of joint subject to brittle fracture when overloaded, are another example of a case where these special load combinations should be used. Other design situations that warrant the use of these equations are noted throughout the *Provisions*.

Although the *Provisions* note the most common cases in which structural overstrength can lead to an undesirable failure mode, it is not possible for them to note all such conditions. Therefore, designers using the *Provisions* should be alert for conditions where the isolated independent failure of any element can lead to a condition of instability or collapse and should use the special load combinations of Eq. 5.2.7.1-1 and 5.2.7.1-2 for the design of these elements. Other conditions which may warrant such a design approach, although not specifically noted in the *Provisions*, include the design of transfer structures beneath discontinuous lateral force resisting elements; and the design of diaphragm force collectors to shear walls and braced frames, when these are the only method of transferring force to these elements at a diaphragm level.

5.2.8 Deflection and Drift Limits: This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "Story drift" is the maximum lateral displacement within a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads).
2. The lateral displacement or deflection due to design forces is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

There are many reasons for controlling drift; one is to control member inelastic strain. Although use of drift limitations is an imprecise and highly variable way of controlling strain, this is balanced by the current state of knowledge of what the strain limitations should be.

Stability considerations dictate that flexibility be controlled. The stability of members under elastic and inelastic deformation caused by earthquakes is a direct function of both axial loading and bending of members. A stability problem is resolved by limiting the drift on the vertical load carrying elements and the resulting secondary moment from this axial load and deflection (frequently called the *P*-delta effect). Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from the *P*-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic safety elements. Since general damage control for economic reasons is not a goal of this document and since the state of the art is not well developed in this area, the drift limits have been established without regard to considerations such as present worth of future repairs versus additional structural costs to limit drift. These are matters for building owners and designers to examine. To the extent that life might be excessively threatened, general nonstructural damage to nonstructural and seismic safety elements is a drift limit consideration.

The design story drift limits of Table 5.2.8. reflect consensus judgment taking into account the goals of drift control outlined above. In terms of life safety and damage control objectives, the drift limits should yield a substantial, though not absolute, measure of safety for well detailed and constructed brittle elements and provide tolerable limits wherein the seismic safety elements can successfully perform, provided they are designed and constructed in accordance with these *Provisions*.

To provide a higher performance standard, the drift limit for the essential facilities of Seismic Use Group III is more stringent than the limit for Groups I and II except for masonry shear wall buildings.

The drift limits for low-rise structures are relaxed somewhat provided the interior walls, partitions, ceilings, and exterior wall systems have been designed to accommodate story drifts. The type of steel building envisioned by the exception to the table would be similar to a prefabricated steel structure with metal skin. When the more liberal drift limits are used, it is recommended that special requirements be provided for the seismic safety elements to accommodate the drift.

It should be emphasized that the drift limits, Δ_a, of Table 5.2.8. are story drifts and, therefore, are applicable to each story (i.e., they must not be exceeded in any story even though the drift in other stories may be well below the limit.) The limit, Δ_a is to be compared to the design story drift as determined by Sec. 5.4.6.1.

Stress or strength limitations imposed by design level forces occasionally may provide adequate drift control. However, it is expected that the design of moment resisting frames, especially steel building frames, and the design of tall, narrow shear wall or braced frame buildings will be governed at least in part by drift considerations. In areas having large design spectral response accelerations, S_{DS} and S_{D1}, it is expected that seismic drift considerations will predominate for buildings of medium height. In areas having a low design spectral response accelerations and for very tall buildings in areas with large design spectral response accelerations , wind considerations generally will control, at least in the lower stories.

Due to probable first mode drift contributions, the Sec. 5.3 ELF procedure may be too conservative for drift design of very tall moment-frame buildings. It is suggested for these buildings, where the first mode would be responding in the constant displacement region of a response spectra (where displacements would be essentially independent of stiffness), that the modal analysis procedure of Sec. 5.5 be used for design even when not required by Sec. 5.2.5.

Building separations and seismic joints are separations between two adjoining buildings or parts of the same building, with or without frangible closures, for the purpose of permitting the adjoining buildings or parts to respond independently to earthquake ground motion. Unless all portions of the structure have been designed and constructed to act as a unit, they must be separated by seismic joints. For irregular structures that cannot be expected to act reliably as a unit, seismic joints should be utilized to separate the building into units whose independent response to earthquake ground motion can be predicted.

Although the *Provisions* do not give precise formulations for the separations, it is required that the distance be "sufficient to avoid damaging contact under total deflection" in order to avoid interference and possible destructive hammering between buildings. It is recommended that the distance be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing separations with height). If the effects of hammering can be shown not to be detrimental, these distances can be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, it is suggested that older code requirements for structural separations of at least 1 in. (25 mm) plus ½ in. (13 mm) for each 10 ft (3 m) of height above 20 ft (6 m) be followed.

5.3 INDEX FORCE ANALYSIS PROCEDURE: This analysis procedure, which was added to the *Provisions* in the 1997 edition, is applicable only to *structures* in *Seismic Design Category* A. Such *structures* are not designed for resistance to any specific level of earthquake ground shaking as the probability that they would ever experience shaking of sufficient intensity to cause life threatening damage is very low so long as the structures are designed with basic levels of structural integrity. Minimum levels of structural integrity are achieved in a *structure* by assuring that all elements in the structure are tied together so that the *structure* can respond to shaking demands in an integral manner and also by providing the *structure* with a complete *seismic-force-resisting system*. It is believed that structures having this level of integrity would be able to resist, without collapse, the very infrequent earthquake ground shaking that could affect them. In addition, requirements to provide such integrity provides collateral benefit with regard to the ability of the structure to survive other hazards such as high wind storms, tornadoes, and hurricanes.

The index force analysis procedure is intended to be a simple approach to ensuring both that a building has a complete *seismic force-resisting-system* and that it is capable of sustaining at least a minimum level of lateral force. In this analysis procedure, a series of static lateral forces equal to 1 percent of the weight at each level of the structure is applied to the structure independently in each of two orthogonal directions. The structural elements of the *seismic-force-resisting system* then are designed to resist the resulting forces in combination with other loads under the load combinations specified by the building code.

The selection of 1 percent of the building weight as the design force for *Seismic Design Category* A *structures* is somewhat arbitrary. This level of design lateral force was chosen as being consistent with prudent requirements for lateral bracing of *structures* to prevent inadvertent buckling under gravity loads and also was believed to be sufficiently small as to not present an undue burden on the design of *structures* in zones of very low seismic activity.

The gravity load W is the total weight of the building and that part of the service load that might reasonably be expected to be attached to the building at the time of an earthquake. It includes permanent and movable partitions and permanent equipment such as mechanical and electrical equipment, piping, and ceilings. The normal human live load is taken to be negligibly small in its contribution to the seismic lateral forces. Buildings designed for storage or warehouse usage should have at least 25 percent of the design floor live load included in the weight, W. Snow loads up to 30 psf (1400 Pa) are not considered. Freshly fallen snow would have little effect on the lateral force in an earthquake; however, ice loading would be more or less firmly attached to the roof of the building and would contribute significantly to the inertia force. For this reason, the effective snow load is taken as the full snow load for those regions where the snow load exceeds 30 psf with the proviso that the local authority having jurisdiction may allow the snow load to be reduced up to 80 percent. The question of how much snow load should be included in W is really a question of how much ice buildup or snow entrapment can be expected for the roof configuration or site topography, and this is a question best left to the discretion of the local authority having jurisdiction.

5.4 EQUIVALENT LATERAL FORCE PROCEDURE: This section discusses the equivalent lateral force (ELF) procedure for seismic analysis of structures.

5.4.1 Seismic Base Shear: The heart of the ELF procedure is Eq. 5.4.1.-1 for base shear, which gives the total seismic design force, V, in terms of two factors: a seismic response coefficient, C_s, and the total gravity load of the building, W. The seismic response coefficient C_s, is obtained from Eq. 5.4.1.1-1 and 5.4.1.1-2 based on the design spectral response accelerations, S_{DS} and S_{D1}. These acceleration parameters and the derivation of the response spectrum is discussed more fully in the *Commentary* for Chapter 4.

The base shear formula and the various factors contained therein were arrived at as explained below.

Elastic Acceleration Response Spectra: See the *Commentary* for Chapter 4 for a full discussion of the shape of the spectra accounting for dynamic response amplification and the effect of site response.

Elastic Design Spectra: The elastic acceleration response spectra for earthquake motions has a descending branch for longer values of T, the period of vibration of the system, that varies roughly as $1/T$. In previous editions of the *Provisions*, the actual response spectra that varied in a 1/T relationship were replaced with design spectra that varied in a $1/T^{2/3}$ relationship. This was intentionally done to provide added conservatism in the design of tall structures, as well as to account for the effects of higher mode participation. In the development of the 1997 *Provisions*, a special task force, known as the Seismic Design Procedures Group (SDPG), was convened to develop a method for using new seismic hazard maps, developed by the USGS in the *Provisions*. Whereas older seismic hazard maps provided an effective peak ground acceleration coefficient C_a and an effective peak velocity related acceleration coefficient C_v, the new maps directly provide parameters that correspond to points on the response spectrum. It was the recommendation of the SDPG that the true shape of the response spectrum, represented by a 1/T relationship, be maintained in the base shear equation. In order to maintain the added conservatism for tall and high occupancy structures, formerly provided by the design spectra which utilized a $1/T^{2/3}$ relationship, the 1997 *Provisions* adopted an occupancy importance factor I into the base shear equation. This I factor, which has a value of 1.25 for Seismic Use Group II structures and 1.5 for Seismic Use Group III structures has the effect of raising the design spectrum for taller, high occupancy structures, to levels comparable to those for which they were designed in pervious editions of the *Provisions*.

Although the introduction of an occupancy importance factor in the 1997 edition adjusted the base shear to more conservative values for large buildings with higher occupancies, it did not address the issue of accounting for higher mode effects, which can be significant in longer period structures, with fundamental modes of vibration significantly larger than the period T_s, at which the response spectrum changes from one of constant response acceleration (Eq. 5.4.1.1-1) to one of constant response velocity (eq. 5.4.1.1-2).

Equation 5.4.1.1-2 could be modified to produce an estimate of base shear that is more consistent with the results predicted by elastic response spectrum methods. Some suggestions for such modifications may be found in Chopra (1995). However, it is important to note that even if the base shear equation were to more accurately simulate results of an elastic response spectrum analysis, most structures respond to design level ground shaking in an inelastic manner. This inelastic response results in different demands than are predicted by elastic analysis, regardless of how "exact" the analysis is. Inelastic response behavior in multistory buildings could be partially accounted for by other modifications to the seismic coefficient C_s. Specifically, the coefficient could be made larger to limit the ductility demand in multistory buildings to the same value as for SDF systems. Results supporting such an approach may be found in (Chopra, 1995) and in (Nassar and Krawinkler, 1991).

The above notwithstanding, the equivalent lateral force procedure is intended to provide a relatively straight forward design approach where complex analyses, accurately accounting for dynamic and inelastic response effects, are not warranted. Rather than making the procedure more complex, so that it would be more appropriate for structures with significant higher mode response, in the 2000 edition of the *Provisions*, it was elected to limit the application of this technique in Seismic Design Categories D, E, and F to those structures where higher mode effects are not significant. Given the widespread use of computer-assisted analysis for major structures, it was felt that these limitations on the application of the equivalent lateral force technique would not be burdensome. It should be noted that particularly for tall structures, the use of dynamic analysis methods will not only result in a more realistic characterization of the distribution of inertial forces in the structure, but may also result in reduced forces, particularly with regard to overturning demands. Therefore, use of the dynamic analysis methods is recommended for such structures, regardless of the Seismic Design Category

Historically, the ELF analytical approach has been limited in application in Seismic Design Categories D, E, and F to regular structures with heights of 240 ft (70 m) or less and irregular structures with heights of 100 ft (30 m) or less. Following recognition that the use of a base shear equation with a 1/T relationship underestimated the response of structures with significant higher mode participation, a change in the height limit for regular structures to 100 ft (30 m) was contemplated. However, the importance of higher mode participation in structural response is a function both of the *structure's* dynamic properties, which are dependent on height, mass and the stiffness of various lateral force resisting elements, and also the frequency content of the ground shaking, as represented by the response spectrum. Therefore, rather than continuing to use building height as the primary parameter used to control analysis procedures, it was decided to limit the application of the ELF to those *structures* in Seismic Design Categories D, E, and F having fundamental periods of response less than 3.5 times the period at which the response spectrum transitions from constant response acceleration to constant response velocity. This limit was selected based on comparisons of the base shear calculated by the ELF equations to that predicted by response spectrum analysis for structures of various periods on five different sites, representative of typical conditions in the eastern and western United States. For all 5 sites, it was determined that the ELF equations conservatively bound the results of a response spectrum analysis for structures having periods less than the indicated amount.

Response Modification Factor: The factor R in the denominator of Eq. 5.4.1.1-1 and 5.4.1.1-2 is an empirical response reduction factor intended to account for damping, overstrength and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Thus, for a lightly damped building structure of brittle material that would be unable to tolerate any appreciable deformation beyond the elastic range, the factor R would be close to 1 (i.e., no reduction from the linear elastic response would be allowed). At the other extreme, a heavily damped building structure with a very ductile structural system would be able to withstand deformations considerably in excess of initial yield and would, therefore, justify the assignment of a larger response reduction factor R. Table 5.2.2 in the *Provisions* stipulates R coefficients for different types of building systems using several different structural materials. The coefficient R ranges in value from a minimum of 1-1/4 for an unreinforced masonry bearing wall system to a maximum of 8 for a special moment frame system. The basis for the R factor values specified in Table 5.2.2 is explained in the Sec. 5.2.1.

The effective value of R used in the base shear equation is adjusted by the occupancy importance factor I. The I value, which ranges from 1 to 1.5, has the effect of reducing the amount of ductility the structure will be called on to provide at a given level of ground shaking. However, it must be recognized that added strength, by itself, is not adequate to provide for superior seismic performance in buildings with critical occupancies. Good connections and construction details, quality assurance procedures, and limitations on building deformation or drift are also important to significantly improve the capability for maintenance of function and safety in critical facilities and those with a high-density occupancy. Consequently, the reduction in the damage potential of critical facilities (Group III) is also handled by using more conservative drift controls (Sec. 5.2.8.) and by providing special design and detailing requirements (Sec. 5.2.6) and materials limitations (Chapters 8 through 12).

5.4.2 Period Determination: In the denominator of Eq. 5.4.1.1-2, T is the fundamental period of vibration of the building. It is preferable that this be determined using modal analysis methods and the principals of structural mechanics. However, methods of structural mechanics cannot be employed to calculate the vibration period before a building has been designed. Consequently, this section provides an approximate method that can be used to estimate building period, with minimal information available on the building design. It is based on the use of simple formulas that involve only a general description of the building type (e.g., steel moment frame, concrete moment frame, shear wall system, braced frame) and overall dimensions (e.g., height and plan length) to estimate the vibration period in order to calculate an initial base shear and proceed with a preliminary design. It is advisable that this base shear and the corresponding value of T be conservative. Even for final design, use of a large value for T is unconservative. Thus, the value of *T used in design* should be smaller than the period calculated for the bare frame of the building. Equations 5.4.2.1-1, 5.4.2.1-2, and 5.4.2.1-3 for the approximate period T_a are therefore intended to provide conservative estimates of the fundamental period of vibration. An upper bound is placed on the value of T calculated using more exact methods, based on T_a and the factor C_u. The coefficient C_u accommodates the likelihood that buildings in areas with lower lateral force requirements probably will be more flexible. Furthermore, it results in less dramatic changes from present practice in lower risk areas. It is generally accepted that the *empirical* equations for T_a are tailored to fit the type of construction common in areas with high lateral force requirements. It is unlikely that buildings in lower risk seismic areas would be designed to produce as high a drift level as allowed in the *Provisions* due to stability problems (P-delta) and wind requirements. For buildings whose design are actually "controlled" by wind, the use of a large T will not really result in a lower design force; thus, use of this approach in high-wind regions should not result in unsafe design.

Taking the seismic base shear to vary as $1/T$ and assuming that the lateral forces are distributed linearly over the height and the deflections are controlled by drift limitations, a simple analysis of the vibration period by Rayleigh's method leads to the conclusion that the vibration period of moment resisting frame structures varies roughly as $h_n^{3/4}$ where h_n equals the total height of the building as defined elsewhere. Based on this, for many years Eq. 5.3.3.1-1 appeared in the *Provisions* in the form:

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

A large number of strong motion instruments have been placed in buildings located within zones of high seismic activity by the U.S. Geological Survey and the California Division of Mines and

Geology. Over the past several years, this has allowed the response of a significant number of these buildings to strong ground shaking to be recorded and the fundamental period of vibration of the buildings to be calculated. Figures C5.4.2.1-1, C5.4.2.1-2, and C5.4.2.1-3, respectively, show plots of these data as a function of building height for three classes of structures. Figure C.5.4.2.1-1 shows the data for moment-resisting concrete frame buildings; Figure C.5.4.2.1-2, for moment-resisting steel frame buildings; and Figure C.5.4.2.1-3, for concrete shear wall buildings. Also shown in these figures are equations for lines that envelop the data within approximately a standard deviation above and below the mean. For the 2000 *Provisions*, Eq, 5.4.2.1-1 is revised into a more general form allowing the statistical fits of the data shown in the figures to be used directly. The values of the coefficient C_t and the superscript x given in Table 5.4.2.1 for these moment-resisting frame structures represent the lower bound (mean -1s) fits to the data shown in Figures C5.4.2.1-1 and C.5.4.2.1-2, respectively, for steel and concrete moment frames. Although updated data were available for concrete shear wall strucures, these data do not fit well with an equation of the form of Eq. 5.4.2.1-1. This is because the period of shear wall buildings is highly dependent not only on the height of the structure but also on the amount of shear wall present in the building. Analytical evaluations performed by Chopra and Goel (1997 and 1998) indicate that equations of the form of Eq. 5.4.2.1-3, 5.4.2.1-4, and 5.4.2.1-5 provide a reasonably good fit to the data. However, the form of these equations is somewhat complex. Therefore, the simpler form of Eq. 5.4.2.1 contained in earlier editions of the *Provisions* was retained with the newer, more accurate formulation presented as an alternative formulation.

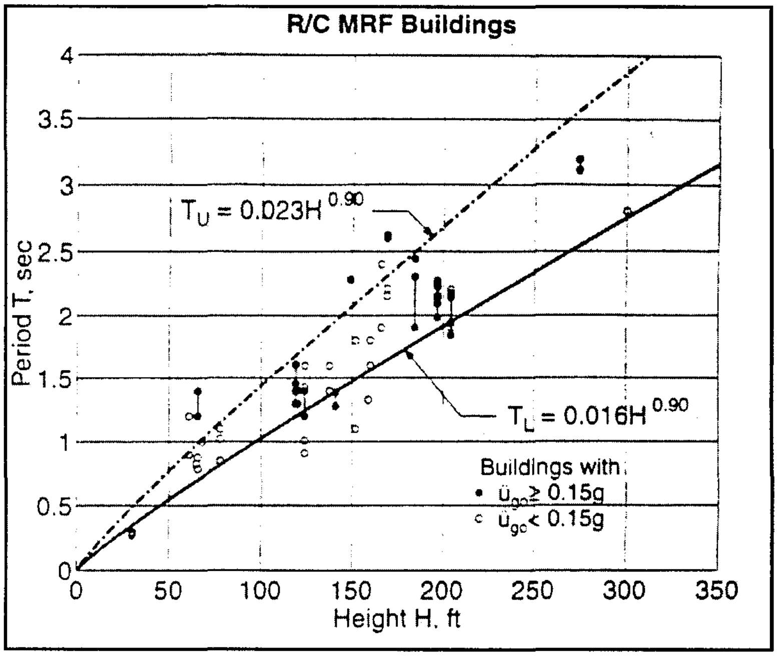


Figure C5.4.2.1-1 Measured building period for reinforced concrete frame structures.

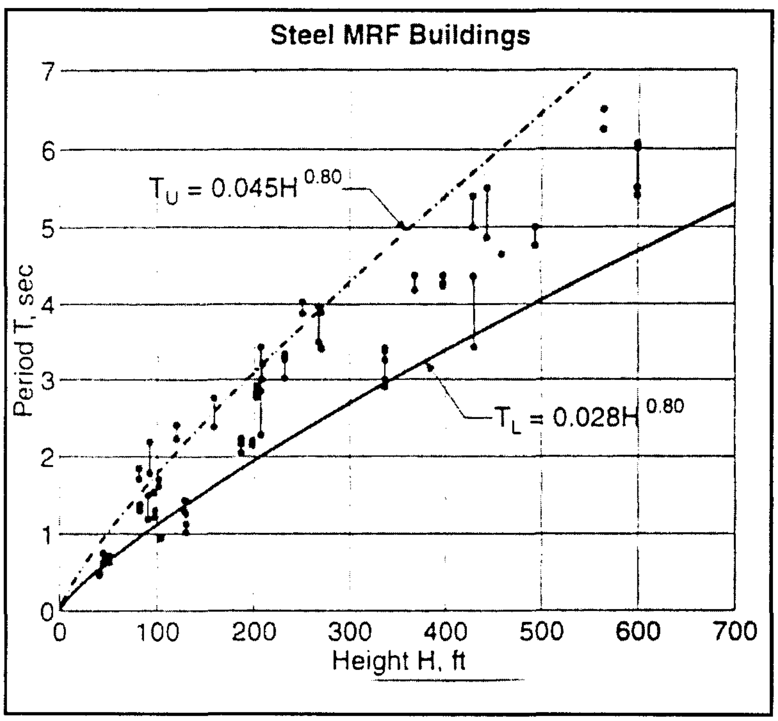


Figure C5.4.2.1-2 Measured building period for moment-resisting steel frame structures.

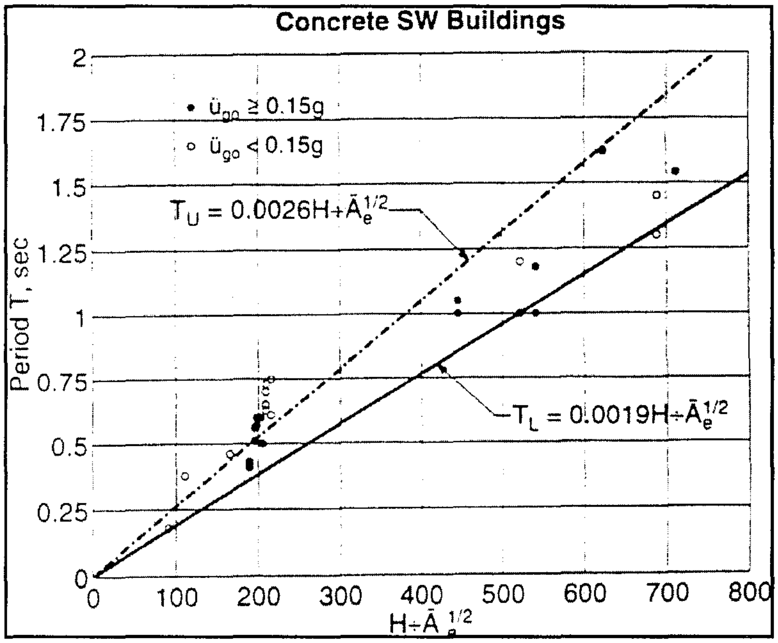


Figure C5.4.2.1-3 Measured building period for concrete shear wall structures.

Updated data for other classes of construction were not available. As a result, the C_t and x values for other types of construction shown in Table 5.4.2.1 are values largely based on limited data obtained from the 1971 San Fernando earthquake that have traditionally been used in the *Provisions*. The optional use of $T = 0.1N$ (Eq. 5.4.2.1-2) is an approximation for low to moderate height frames that has been long in use.

As an exception to Eq. 5.4.2.1-1, these requirements allow the calculated fundamental period of vibration, T, of the seismic-force-resisting system to be used in calculating the base shear. However, the period, T, used may not exceed $C_u T_a$ with T_a determined from Eq. 5.4.2.1-1.

In earlier editions of the *Provisions*, the C_u coefficient varied from a value of 1.2 in zones of high seismicity to a value of 1.7 in zones of low seismicity. The data presented in Figures C5.4.2.1-1, C5.4.2.1-2, and C5.4.2.1-3 permit direct evaluation of the upper bound on period as a function of the lower bound, given by Eq. 5.4.2.1-1. This data indicates that in zones of high seismicity, the ratio of the upper to lower bound may more properly be taken as a value of about 1.4. Therefore, in the 2000 *Provisions*, the values in Table 5.4.2 were revised to reflect this data in zones of high seismicity while retaining the somewhat subjective values contained in earlier editions for the zones of lower seismicity.

For exceptionally stiff or light buildings, the calculated T for the seismic-force-resisting system may be significantly shorter than T_a calculated by Eq. 5.4.2.1-1. For such buildings, it is recommended that the period value T be used in lieu of T_a for calculating the seismic response coefficient, C_s.

Although the approximate methods of Sec. 3.3.3. can be used to determine a period for the design of structures, the fundamental period of vibration of the seismic-force-resisting system should be calculated according to established methods of mechanics. Computer programs are available for such calculations. One method of calculating the period, probably as convenient as any, is the use of the following formula based on Rayleigh's method:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \tag{C5.4.2}$$

where:

F_i = the seismic lateral force at Level i,

w_i = the gravity load assigned in Level i,

d_i = the static lateral displacement at Level i due to the forces F_i computed on a linear elastic basis, and

g = is the acceleration of gravity.

The calculated period increases with an increase in flexibility of the structure because the d term in the Rayleigh formula appears to the second power in the numerator but to only the first power in the denominator. Thus, if one ignores the contribution of nonstructural elements to the stiffness of the structure in calculating the deflections d, the deflections are exaggerated and the calculated period is lengthened, leading to a decrease in the seismic response coefficient C_s and, therefore, a decrease in the design force. Nonstructural elements do not know that they are non-structural. They participate in the behavior of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side. The limitation of $C_u T_a$ is imposed as a safeguard.

5.4.3 Vertical Distribution of Seismic Forces: The distribution of lateral forces over the height of a structure is generally quite complex because these forces are the result of superposition of a number of natural modes of vibration. The relative contributions of these vibration modes to the total forces depends on a number of factors including the shape of the earthquake response spectrum, the natural periods of vibration of the structure, and the shapes of vibration modes that, in turn, depend on the mass and stiffness over the height (see Sec. 5.2.3). The basis of this method is discussed below. In structures having only minor irregularity of mass or stiffness over the height, the accuracy of the lateral force distribution as given by Eq. 5.4.3-2 is much improved by the procedure described in the last portion of Sec. 5.2.4 of this commentary. The lateral force at each level, x, due to response in the first (fundamental) natural mode of vibration is:

$$f_{x1} = V_1 \left(\frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_i \phi_{i1}} \right) \tag{C5.4.3}$$

where:

V_1 = the contribution of this mode to the base shear,

w_i = the weight lumped at the ith level, and

ϕ_i = the amplitude of the first mode at the i^{th} level.

This is the same as Eq. 5.5.5-2 in Sec. 5.5 of the *Provisions*, but it is specialized for the first mode. If V_1 is replaced by the total base shear, V, this equation becomes identical to Eq. 5.4.3-2 with $k = 1$ if the first mode shape is a straight line and with $k = 2$ if the first mode shape is a parabola with its vertex at the base.

It is well known that the influence of modes of vibration higher than the fundamental mode is small in the earthquake response of short period structures and that, in regular structures, the fundamental vibration mode departs little from a straight line. This, along with the matters discussed above, provides the basis for Eq. 5.3.4-2 with $k = 1$ for structures having a fundamental vibration period of 0.5 seconds or less.

It has been demonstrated that although the earthquake response of long period structures is primarily due to the fundamental natural mode of vibration, the influence of higher modes of vibration can be significant and, in regular structures, the fundamental vibration mode lies approximately between a straight line and a parabola with the vertex at the base. Thus, Eq. 5.3.4-2 with $k = 2$ is appropriate for structures having a fundamental period of vibration of 2.5 seconds or longer. Linear variation of k between 1 at a 0.5 second period and 2 at a 2.5 seconds period provides the simplest possible transition between the two extreme values.

5.4.4 Horizontal Shear Distribution: The story shear in any story is the sum of the lateral forces acting at all levels above that story. Story x is the story immediately below Level x (Figure C5.4.4). Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm. Similarly, the stiffness of moment or braced frames will establish the distribution of the story shear to the vertical resisting elements in that story.

5.4.4.1 and 5.4.4.2 Inherent and Accidental Torsion: The torsional moment to be considered in the design of elements in a story consists of two parts:

1. M_t, the moment due to eccentricity between centers of mass and resistance for that story, is to be computed as the story shear times the eccentricity perpendicular to the direction of applied earthquake forces.

2. M_{ta}, commonly referred to as "accidental torsion," is to be computed as the story shear times the "accidental eccentricity," equal to 5 percent of the dimension of the structure, in the story under consideration perpendicular to the direction of the applied earthquake forces.

Computation of M_{ta} in this manner is equivalent to the procedure in Sec. 5.4.4 which implies that the dimension of the structure is the dimension in the story where the torsional moment is being computed and that all the masses above that story should be assumed to be displaced in the same direction at one time (e.g., first, all of them to the left and, then, to the right).

Dynamic analyses assuming linear behavior indicate that the torsional moment due to eccentricity between centers of mass and resistance may significantly exceed M_t (Newmark and Rosenblueth, 1971). However, such dynamic magnification is not included in the *Provisions*, partly because its significance is not well understood for structures designed to deform well beyond the range of linear behavior.

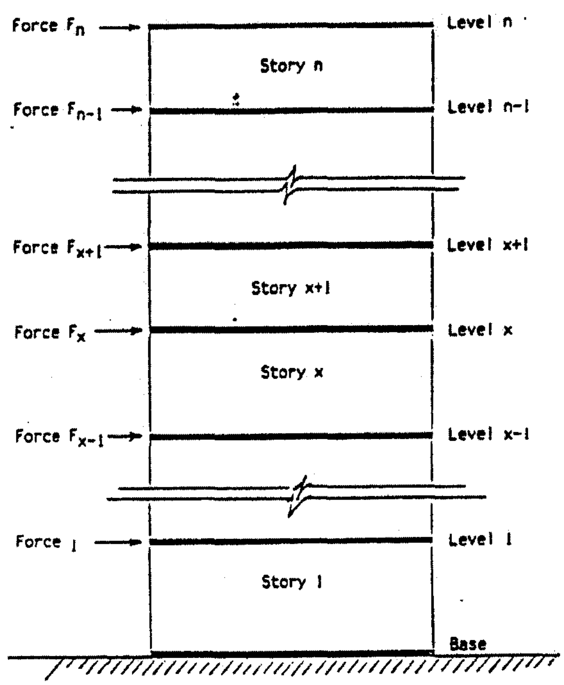


FIGURE C5.4.4 Description of story and level. The shear at Story *x* (*Vx*) is the sum of all the lateral forces at and above Story *x* (F_x through F_n).

The torsional moment M_t calculated in accordance with this provision would be zero in those stories where centers of mass and resistance coincide. However, during vibration of the structure, torsional moments would be induced in such stories due to eccentricities between centers of mass and resistance in other stories. To account for such effects, it is recommended that the torsional moment in any story be not smaller than the following two values (Newmark and Rosenblueth, 1971):

1. The story shear times one-half of the maximum of the computed eccentricities in all stories below the one being analyzed and
2. One-half of the maximum of the computed torsional moments for all stories above.

Accidental torsion is intended to cover the effects of several factors that have not been explicitly considered in the *Provisions*. These factors include the rotational component of ground motion about a vertical axis; unforeseeable differences between computed and actual values of stiffness, yield strengths, and dead-load masses; and unforeseeable unfavorable distributions of dead- and live-load masses.

There are indications that the 5 percent accidental eccentricity may be too small in some structures since they may develop torsional dynamic instability. Some examples are the upper stories of tall structures having little or no nominal eccentricity, those structures where the calculations of relative stiffnesses of various elements are particularly uncertain (e.g., those that depend largely on masonry walls for lateral force resistance or those that depend on vertical elements made of different materials), and nominally symmetrical structures that utilize core elements alone for seismic resistance or that behave essentially like elastic nonlinear systems (e.g., some prestressed concrete frames). The amplification factor for torsionally irregular structures (Eq. 5.4.4.1.3-1) was introduced in the 1988 Edition as an attempt to account for some of these problems in a controlled and rational way.

The way in which the story shears and the effects of torsional moments are distributed to the vertical elements of the seismic-force-resisting system depends on the stiffness of the diaphragms relative to vertical elements of the system.

Where the diaphragm stiffness in its own plane is sufficiently high relative to the stiffness of the vertical *components* of the system, the diaphragm may be assumed to be indefinitely rigid for purposes of this section. Then, in accordance with compatibility and equilibrium requirements, the shear in any story is to be distributed among the vertical *components* in proportion to their contributions to the lateral stiffness of the story while the story torsional moment produces additional shears in these *components* that are proportional to their contributions to the torsional stiffness of the story about its center of resistance. This contribution of any *component* is the product of its lateral stiffness and the square of its distance to the center of resistance of the story. Alternatively, the story shears and torsional moments may be distributed on the basis of a three-dimensional analysis of the structure, consistent with the assumption of linear behavior.

Where the diaphragm in its own plane is very flexible relative to the vertical *components*, each vertical *component* acts almost independently of the rest. The story shear should be distributed to the vertical *components* considering these to be rigid supports. Analysis of the diaphragm acting as a continuous horizontal beam or truss on rigid supports leads to the distribution of shears. Because the properties of the beam or truss may not be accurately computed, the shears in vertical elements should not be taken to be less than those based on "tributary areas." Accidental torsion may be accounted for by adjusting the position of the horizontal force with respect to the supporting vertical elements.

There are some common situations where it is obvious that the diaphragm can be assumed to be either rigid or very flexible in its own plane for purposes of distributing story shear and considering torsional moments. For example, a solid monolithic reinforced concrete slab, square or nearly square in plan, in a structure with slender moment resisting frames may be regarded as rigid. A large plywood diaphragm with widely spaced and long, low masonry walls may be regarded as very flexible. In intermediate situations, the design forces should be based on an analysis that explicitly considers diaphragm deformations and satisfies equilibrium and compatibility requirements. Alternatively, the design forces should be the envelope of the two sets of forces resulting from both extreme assumptions regarding the diaphragms--rigid or very flexible.

Where the horizontal diaphragm is not continuous, the story shear can be distributed to the vertical *components* based on their tributary areas.

5.4.5 Overturning: This section requires that the structure be designed to resist overturning moments statically consistent with the design story shears. In the 1997 and earlier editions of the provisions, the overturing moment was modified by a factor, τ, to account in an approximate manner, for the effects of higher mode response in taller structures. In the 2000 edition of the *Provisions*, the equivalent lateral force technique was limited in application in Seismic Design Categories D, E, and F to structures that do not have significant higher mode participation. As a result it was no longer necessary to include this τ coefficient for these structures permitting a significant simplification in the design procedures. Under this new approach tall structures in Seismic Design Categories B and C designed using the equivalent lateral force procedure will be designed for somewhat larger overturning demands than under past editions of the Provisions. This conservatism was accepted as an inducement for designers of such structures to use the more appropriate dynamic analysis procedure.

In the design of the foundation, the overturning moment calculated at the foundation-soil interface may be reduced to 75 percent of the calculated value using Eq. 5.4.1-1. This is appropriate because a slight uplifting of one edge of the foundation during vibration leads to reduction in the overturning moment and because such behavior does not normally cause structural distress.

5.4.6 Drift Determination and *P*-delta Effects: This section defines the design story drift as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe} (determined from an elastic analysis), by the deflection amplification factor, C_d, given in Table 5.2.2. The elastic analysis is to be made for the seismic-force-resisting system using the prescribed seismic design forces and considering the structure to be fixed at the base. Stiffnesses other than those of the seismic-force-resisting system should not be included since they may not be reliable at higher inelastic strain levels.

The deflections are to be determined by combining the effects of joint rotation of members, shear deformations between floors, the axial deformations of the overall lateral resisting elements, and the shear and flexural deformations of shear walls and braced frames. The deflections are determined initially on the basis of the distribution of lateral forces stipulated in Sec. 5.4.3. For frame structures, the axial deformations from bending effects, although contributing to the overall structural distortion, may or may not affect the story-to-story drift; however, they are to be considered. Centerline dimensions between the frame elements often are used for analysis, but clear span dimensions with consideration of joint panel zone deformation also may be used.

For determining compliance with the story drift limitation of Sec. 5.2.7, the deflections, δ_x, may be calculated as indicated above for the seismic-force-resisting system and design forces corresponding to the fundamental period of the structure, T (calculated without the limit $T \le C_u T_a$ specified in Sec. 5.4.2), may be used. The same model of the seismic-force-resisting system used in determining the deflections must be used for determining T. The waiver does not pertain to the calculation of drifts for determining P-delta effects on member forces, overturning moments, etc. If the P-delta effects determined in Sec. 5.4.6.2 are significant, the design story drift must be increased by the resulting incremental factor.

The P-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces prescribed in Sec. 5.4.3 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the *P*-delta moment to the lateral force story moment is designated as a stability coefficient, θ, in Eq. 5.4.6.2-1. If the stability coefficient θ is less than 0.10 for every story, the *P*-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient θ exceeds 0.10 for any story, the *P*-delta effects on story drifts, shears, member forces, etc., for the whole structure must be determined by a rational analysis.

An acceptable *P*-delta analysis, based upon elastic stability theory, is as follows:

1. Compute for each story the *P*-delta amplification factor, $a_d = \theta/(1 - \theta)$. a_d takes into account the multiplier effect due to the initial story drift leading to another increment of drift that would lead to yet another increment, etc. Thus, both the effective shear in the story and the computed eccentricity would be augmented by a factor $1 + \theta + \theta^2 + \theta^3$ ..., which is $1/(1 - \theta)$ or $(1 + a_d)$.
2. Multiply the story shear, V_x, in each story by the factor $(1 + a_d)$ for that story and recompute the story shears, overturning moments, and other seismic force effects corresponding to these augmented story shears.

This procedure is applicable to planar structures and, with some extension, to three-dimensional structures. Methods exist for incorporating two- and three-dimensional *P*-delta effects into computer analyses that do not explicitly include such effects (Rutenburg, 1985). Many programs explicitly include *P*-delta effects. A mathematical description of the method employed by several popular programs is given by Wilson and Habibullah (1987).

The *P*-delta procedure cited above effectively checks the static stability of a structure based on its initial stiffness. Since the inception of this procedure with ATC 3-06, however, there has been some debate regarding its accuracy. This debate stems from the intuitive notion that the structure's secant stiffness would more accurately represent inelastic *P*-delta effects. Given the additional uncertainty of the effect of dynamic response on *P*-delta behavior and the (apparent) observation that instability-related failures rarely occur in real structures, the *P*-delta requirements remained as originally written until revised for the 1991 Edition.

There was increasing evidence that the use of inelastic stiffness in determining *theoretical* *P*-delta response is unconservative. Given a study carried out by Bernal (1987), it was argued that *P*-delta amplifiers should be based on secant stiffness and that, in other words, the C_d term in Eq.5.4.6.2-1 should be deleted. However, since Bernal's study was based on the inelastic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties existed regarding the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

Another problem with accepting a *P*-delta procedure based on secant stiffness was that design forces would be greatly increased. For example, consider an ordinary moment frame of steel with a C_d of 4.0 and an elastic stability coefficient θ of 0.15. The amplifier for this structure would be 1.0/0.85 = 1.18 according to the 1988 Edition of the *Provisions*. If the *P*-delta effects were based on secant stiffness, however, the stability coefficient would increase to 0.60 and the amplifier would become 1.0/0.4 = 2.50. (Note that the 0.9 in the numerator of the amplifier equation in the 1988 Edition was dropped for this comparison.) This example illustrates that there could be an extreme impact on the requirements if a change was implemented that incorporated *P*-delta amplifiers based on static secant stiffness response.

There was, however, some justification for retaining the *P*-delta amplifier as based on elastic stiffness. This justification was the apparent lack of stability-related failures. The reasons for the lack of observed failures included:

1. Many structures display strength well above the strength implied by code-level design forces (see Figure C5.5.1-1). This overstrength likely protects structures from stability-related failures.2.The likelihood of a stability failure decreases with increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most structures designed for extreme ground motion is significantly greater than the stiffness of the same structure designed for lower intensity shaking or for wind. Since damaging low-intensity earthquakes are somewhat rare, there would be little observable damage.

Due to the lack of stability-related failures, therefore, the requirements of the 1988 Edition of the *Provisions* regarding *P*-delta amplifiers remain in the 1991 and 1994 Editions with the exception that the 0.90 factor in the numerator of the amplifier has been deleted. This factor originally was used to create a transition from cases where *P*-delta effects need not be considered ($\theta \leq 0.10$, amplifier = 1.0) to cases where such effects need be considered ($\theta > 1.0$, amplifier > 1.0).

However, the 1991 Edition introduced a requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$, where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 8 through 14 of the *Provisions.*

The purpose of this requirement is to protect structures from the possibility of stability failures triggered by post-earthquake residual deformation. The danger of such failures is real and may not be eliminated by apparently available overstrength. This is particularly true of structures designed in regions of lower seismicity.

The computation of θ_{max}, which, in turn, is based on βC_d, requires the computation of story strength supply and story strength demand. Story strength demand is simply the seismic design shear for the story under consideration. The story strength supply may be computed as the shear in the story that occurs simultaneously with the attainment of the development of first significant yield of the overall structure. To compute first significant yield, the structure should be loaded with a seismic force pattern similar to that used to compute seismic story strength demand. A simple and conservative procedure is to compute the ratio of demand to strength for each member of the seismic-force-resisting system in a particular story and then use the largest such ratio as β. For a structure otherwise in conformance with the *Provisions*, $\beta = 1.0$ is obviously conservative.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of added stiffness for drift control, from code-required wind resistance, or simply a feature of other aspects of the design. β = story shear demand/story shear capacity is conservatively 1.0 for any design that meets the remainder of the *Provisions*. Some structures inherently possess more strength than required, but instability is not typically a concern for such structures. For many flexible structures, the proportions of the structural members are controlled by the drift requirements rather than the strength requirements; consequently, β is less than 1.0 because the members provided are larger and stronger than required. This has the effect of reducing the inelastic *component* of total seismic drift and, thus, β is placed as a factor on C_d.

Accurate evaluation of β would require consideration of all pertinent load combinations to find the maximum value of seismic load effect demand to seismic load effect capacity in each and every member. A conservative simplification is to divide the total demand with seismic included by the total capacity; this covers all load combinations in which dead and live effects add to seismic. If a member is controlled by a load combination where dead load counteracts seismic, to be correctly computed, the ratio β must be based only on the seismic *component*, not the total; note that the vertical load P in the P-delta computation would be less in such a circumstance and, therefore, θ would be less. The importance of the counteracting load combination does have to be considered, but it rarely controls instability.

5.5 MODAL RESPONSE SPECTRUM ANALYSIS PROCEDURE:

5.5.1 General: Modal analysis (Newmark and Rosenblueth, 1971; Clough and Penzien, 1975; Thomson, 1965; Wiegel, 1970) is applicable for calculating the linear response of complex, multi-degree-of-freedom structures and is based on the fact that the response is the superposition of the responses of individual natural modes of vibration, each mode responding with its own particular pattern of deformation (the mode shape), with its own frequency (the modal frequency), and with its own modal damping. The response of the structure, therefore. can be modeled by the response of a number of single-degree-of-freedom oscillators with properties chosen to be representative of the mode and the degree to which the mode is excited by the earthquake motion. For certain types of damping, this representation is mathematically exact and, for structures, numerous full-scale tests and analyses of earthquake response of structures have shown that the use of modal analysis, with viscously damped single-degree-of-freedom oscillators describing the response of the structural modes, is an accurate approximation for analysis of linear response.

Modal analysis is useful in design. The Equivalent Lateral Force procedure of Sec. 5.4 is simply a first mode application of this technique, that assumes all of the structure's mass is active in the first mode.. The purpose of modal analysis is to obtain the maximum response of the structure in each of its important modes, which are then summed in an appropriate manner. This maximum modal response can be expressed in several ways. For the *Provisions*, it was decided that the modal forces and their distributions over the structure should be given primary emphasis to highlight the similarity to the equivalent static methods traditionally used in building codes (the SEAOC recommendations and the *UBC*) and the ELF procedure in Sec. 5.4. Thus, the coefficient C_{sm} in Eq. 5.5.4-1 and the distribution equations, Eq. 5.5.5-1 and 5.5.5-2, are the counterparts of Eq. 5.4.3-1 and 5.4.3-2. This correspondence helps clarify the fact that the simplified modal analysis contained in Sec. 5.5 is simply an attempt to specify the equivalent lateral forces on a structure in a way that directly reflects the individual dynamic characteristics of the structure. Once the story shears and other response variables for each of the important modes are determined and combined to produce design values, the design values are used in basically the same manner as the equivalent lateral forces given in Sec. 5.4.

5.5.2 Modes: This section defines the number of modes to be used in the analysis. For many structures, including low-rise structures and structures of moderate height, three modes of vibration in each direction are nearly always sufficient to determine design values of the earthquake response of the structure. For high-rise structures, however, more than three modes may be required to adequately determine the forces for design. This section provides a simple rule that the combined participating mass of all modes considered in the analysis should be equal to or greater than 90 percent of the effective total mass in each of two orthogonal horizontal directions.

5.5.3 Modal Properties: Natural periods of vibration are required for each of the modes used in the subsequent calculations. These are needed to determine the modal coefficients C_{sm} from Eqs. 5.5.4. Because the periods of the modes contemplated in these requirements are those associated with moderately large, but still essentially linear, structural response, the period calculations should include only those elements that are effective at these amplitudes. Such periods may be longer than those obtained from a small-amplitude test of the structure when completed or the response to small earthquake motions because of the stiffening effects of nonstructural and architectural *components* of the structure at small amplitudes. During response to strong ground-shaking, however, measured responses of structures have shown that the periods lengthen, indicating the loss of the stiffness contributed by those *components*.

There exists a wide variety of methods for calculation of natural periods and associated mode shapes, and no one particular method is required by the *Provisions*. It is essential, however, that the method used be one based on generally accepted principles of mechanics such as those given in well known textbooks on structural dynamics and vibrations (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Thomson, 1965; Wiegel, 1970). Although it is expected that in many cases computer programs, whose accuracy and reliability are documented and widely recognized, will be used to calculate the required natural periods and associated mode shapes, their use is not required.

5.5.4 Modal Base Shear: A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the structure vibrating in each of its important modes. As the structure vibrates back and forth in a particular mode at the associated period, it experiences maximum values of base shear, interstory drifts, floor displacements, base (overturning) moments, etc. In this section, the base shear in the m^{th} mode is specified as the product of the modal seismic coefficient C_{sm} and the effective weight W_m for the mode. The coefficient C_{sm} is determined for each mode from Eq. 5.5.4-3 using the associated period of the mode, T_m, in addition to the factors C_v and R, which are discussed elsewhere in the *Commentary*. An exception to this procedure occurs for higher modes of those *structures* that have periods shorter than 0.3 second and that are founded on soils of *Site Class* D, E, or F. For such modes, Eq. 5.5.4-4 is used. Equation 5.5.4-4 gives values ranging from $S_{DS}/2.5R$ for very short periods to S_{DS}/R for $T_m = 0.3$. Comparing these values to the limiting values of C_s of S_{DS}/R for soils with Soil Profile Type D as specified following Eq. 5.5.4 -3, it is seen that the use of Eq. 5.5.4-4, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Eq. 5.5.4-3 and its limiting values. The spectral shape so defined is a conservative approximation to average spectra that are known to first ascend, level off, and then decay as period increases. Equation 5.5.4-3 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For soils with Soil Profile Type A, B and C, the ascending portion of the spectra is completed by the time the period reaches a small value near 0.1 or 0.2 second. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5.5.4-4 is then a replacement for the spectral shape for soils with Soil Profile Type D, E and F and short periods that is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 5.5.4-3 for modal analysis in the case of soils with Soil Profile Types D, E, and F. The effective modal gravity load given in Eq. 5.5.4-2 can be interpreted as specifying the portion of the weight of the structure that participates in the vibration of each mode. It is noted that Eq. 5.4.5-2 gives values of W_m that are independent of how the modes are normalized.

The final equation of this section, Eq. 5.5.4-5, is to be used if a modal period exceeds 4 seconds. It can be seen that Eq. 5.5.4-5 and 5.5.4-3 coincide at $T_m = 4$ seconds so that the effect of using Eq. 5.5.4-5 is to provide a more rapid decrease in C_{sm} as a function of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods, the average velocity spectrum of strong earthquake motions from large (magnitude 6.5 and larger) earthquakes is approximately constant, which implies that C_{sm} should decrease as $1/T_m$. For very long periods, the average displacement spectrum of strong earthquake motions becomes constant which implies that C_{sm}, a form of acceleration spectrum, should decay as $1/T_m^2$. The period at which the displacement response spectrum becomes constant depends on the size of the earthquake, being larger for great earthquakes, and a representative period of 4 seconds was chosen to make the transition.

5.5.5 Modal Forces, Deflections, and Drifts: This section specifies the forces and displacements associated with each of the important modes of response.

Modal forces at each level are given by Eq. 5.5.5-1 and 5.5.5-2 and are expressed in terms of the gravity load assigned to the floor, the mode shape, and the modal base shear V_m. In applying the forces F_{xm} to the structure, the direction of the forces is controlled by the algebraic sign of f_{xm}. Hence, the modal forces for the fundamental mode will all act in the same direction, but modal forces for the second and higher modes will change direction as one moves up the structure. The form of Eq. 5.5.5-1 is somewhat different from that usually employed in standard references and shows clearly the relation between the modal forces and the modal base shear. It therefore is a convenient form for calculation and highlights the similarity to Eq. 5.4.3-1 in the ELF procedure.

The modal deflections at each level are specified by Eq. 5.5.5-3. These are the displacements caused by the modal forces F_{xm} considered as static forces and are representative of the maximum amplitudes of modal response for the essentially elastic motions envisioned within the concept of the seismic response modification coefficient R. This is also a logical point to calculate the modal drifts, which are required in Sec. 5.5.7. If the mode under consideration dominates the earthquake response, the modal deflection under the strongest motion contemplated by the *Provisions* can be estimated by multiplying by the deflection amplification factor C_d. It should be noted also that δ_{xm} is proportional to ϕ_{xm} (this can be shown with algebraic substitution for F_{xm} in Eq. 5.5.5-4) and will therefore change direction up and down the structure for the higher modes.

5.5.6 Modal Story Shears and Moments: This section merely specifies that the forces of Eq. 5.5.5-1 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Eq. 5.5.5-1 are applied to each mass, and linear static methods are used to calculate story shears and story overturning moments. The base shear that results from the calculation should check with Eq. 5.5.4-1.

5.5.7 Design Values: This section specifies the manner in which the values of story shear, moment, and drift quantities and the deflection at each level are to be combined. The method used, in which the design value is the square root of the sum of the squares of the modal quantities, was selected for its simplicity and its wide familiarity (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Wiegel, 1970). In general, it gives satisfactory results, but it is not always a conservative predictor of the earthquake response inasmuch as more adverse combinations of modal quantities than are given by this method of combination can occur. The most common instance where combination by use of the square root of the sum of the squares is unconservative occurs when two modes have very nearly the same natural period. In this case, the responses are highly correlated and the designer should consider combining the modal quantities more conservatively (Newmark and Rosenblueth, 1971). In the 1991 Edition of the *Provisions* the option of combining these quantities by the complete quadratic combination (CQC) technique was introduced. This method provides somewhat better results than the square root of the sum of squares method for the case of closely spaced modes.

This section also limits the reduction of base shear that can be achieved by modal analysis compared to use of the ELF procedure. Some reduction, where it occurs, is thought justified because the modal analysis gives a somewhat more accurate representation of the earthquake response. Some limit to any such possible reduction that may occur from the calculation of longer natural periods is necessary because the actual periods of vibration may not be as long, even at moderately large amplitudes of motion, due to the stiffening effects of elements not a part of the seismic resisting system and of nonstructural and architectural *components*. The limit is imposed by comparison to 85 percent of base shear value computed with the ELF procedure. Where modal analysis predicts response quantities with a total base shear less than 85 percent of that which could be computed using the ELF procedure, all response results must be scaled up to that level. Where modal analysis predicts response quantities in excess of those predicted by the ELF procedure, this is likely the result of significant higher mode participation and reduction to the values obtained from the ELF procedure are not permitted.

5.5.8 Horizontal Shear Distribution: This section requires that the design story shears calculated in Sec. 5.5.7 and the torsional moments prescribed in Sec. 5.4.4 be distributed to the vertical elements of the seismic resisting system as specified in Sec. 5.4.4 and as elaborated on in the corresponding section of this commentary.

5.5.9 Foundation Overturning: Because story moments are calculated mode by mode (properly recognizing that the direction of forces F_{xm} is controlled by the algebraic sign of f_{xm}) and then combined to obtain the design values of story moments, there is no reason for reducing these design moments. This is in contrast with reductions permitted in overturning moments calculated from equivalent lateral forces in the analysis procedures of Sec. 5.4 (see Sec. 5.4.5 of this commentary). However, in the design of the foundation, the overturning moment calculated at the foundation-soil interface may be reduced by 10 percent for the reasons mentioned in Sec. 5.4.5 of this commentary.

5.5.10 *P*-Delta Effects: Sec. 5.4.6 of this commentary applies to this section. In addition, to obtain the story drifts when using the modal analysis procedure of Sec. 5.5, the story drift for each mode should be independently determined in each story (Sec. 5.5.5). The story drift should not be determined from the differential combined lateral structural deflections since this latter procedure will tend to mask the higher mode effects in longer period structures.

5.6 LINEAR RESPONSE HISTORY ANALYSIS PROCEDURE: Linear response history analysis, also commonly known as time history analysis, is a numerically complex technique in which the response of a structural model to a specific earthquake ground motion accelerogram is determined through a process of numerical integration of the equations of motion. The ground shaking accelerogram, or record, is digitized into a series of small time steps, typically on the order of 1/100th of a second or smaller. Starting at the initial time step, a finite difference solution, or other numerical integration algorithm is followed to allow the calculation of the displacement of each node in the model and the force in each element of model to be calculated for each time step of the record. For even small structural models, this requires thousands of calculations and produces tens of thousands of data points. Clearly, such a calculation procedure can be performed only with the aid of high speed computers. However, even with the use of such computers, which are now commonly available, interpretation of the voluminous data that results from such analysis is tedious.

The principal advantages of response history analysis, as opposed to response spectrum analysis, is that response history analysis provides a time dependent history of the response of the structure to a specific ground motion, allowing calculation of path dependent effects such as damping and also providing information on the stress and deformation state of the structure throughout the period of response. A response spectrum analysis, however, indicates only the maximum response quantities and does not indicate when during the period of response these occur, or how response of different portions of the structure is phased relative to other portions. Response history analyses are highly dependent on the characteristics of the individual ground shaking record and subtle changes in these records can lead to significant differences with regard to the predicted response of the structure. This is why, when response history analyses are used in the design process, it is necessary to run a suite of ground motion records. The use of multiple records in the analyses allows the difference in response, resulting from differences in record characteristics, to be observed. As a minimum, the *Provisions* require that suites of ground motions include at least three different records. However, suites containing larger numbers of records are preferable, since when more records are run, it is more likely that the differing response possibilities for different ground motion characteristics are observed. In order to encourage the use of larger suites, the *Provisions* require that when a suite contains less than 7 records, the maximum values of the predicted response parameters be used as the design values. When 7 or more records are used, then mean values of the response parameters may be used. This can lead to a substantial reduction in design forces and displacements and typically will justify the use of larger suites of records.

Whenever possible, ground motion records should be scaled form actual recorded earthquake ground motions, obtained from events of similar magnitude to that which controls the design earthquake for the site, and with the instruments being located on sites with similar characteristics and fault distances to that of the building site. Since only a limited number of actual recordings are available for such purposes, the use of synthetic records is permitted and may often be required.

The extra complexity and cost inherent in the use of response history analysis rather than to modal response spectrum analysis is seldom justified and as a result, this procedure is rarely used in the design process. One exception is for the design of *structures* with energy dissipation systems comprised of linear viscous dampers. Linear response history analysis can be used to predict the response of *structures* with such systems, while modal response spectrum analysis can not.

5.7 NONLINEAR RESPONSE HISTORY ANALYSIS: This method of analysis is very similar to linear response history analysis, described in Sec. 5.6 except that the mathematical model is formulated in such a way that the stiffness and even connectivity of the elements can be directly modified based on the deformation state of the structure. This permits the effect of element yielding, buckling and other nonlinear behavior on structural response to be directly accounted for in the analysis. It also permits such nonlinear behaviors as foundation rocking, opening and closing of gaps, nonlinear viscous and hysteric damping to be evaluated. Potentially, this ability to directly account for these various nonlinearities can permit nonlinear response history analysis to provide very accurate evaluations of the response of the structure to strong ground motion. However, this accuracy can seldom be achieved in practice. This is partially because currently available nonlinear models for different elements can only approximate the behavior of real structural elements. Another limit on the accuracy of this approach is the fact that minor deviations in ground motion, such as those described in Sec. 5.6, or even in element hysteric behavior, can result in significant differences in predicted response. For these reasons, when nonlinear response history analysis is used in the design process, suites of ground motion time histories should be considered, as described in Sec. 5.6. It may also be appropriate to perform sensitivity studies, in which the assumed hysteric properties of elements are allowed to vary, within expected bounds, to allow the effects of such uncertainties on predicted response to be evaluated.

Application of nonlinear response history analysis to even the simplest structures requires large, high speed computers and complex computer software that has specifically been developed for this purpose. Several software packages have been in use for this purpose in Universities for a number of years. These include the DRAIN family of programs and also the IDARC and IDARST family of programs. However, these programs have largely been viewed as experimental and are not generally accompanied by the same level of documentation and quality assurance typically found with commercially available software packages typically used in design offices. Although commercial software capable of performing nonlinear response history analyses has been available for several years, the use of these packages has generally been limited to complex aerospace, mechanical and industrial applications.

As a result of this, nonlinear response history analysis has mostly been used as a research, rather than design tool, until very recently. With the increasing adoption of base isolation and energy dissipation technologies in the structural design process, however, the need to apply this analysis technique in the design office has increased, creating a demand for more commercially available software. In response to this demand, several vendors of commercial structural analysis software have modified their analysis programs to include limited nonlinear capability including the ability to model base isolation bearings, viscous dampers, and friction dampers. Some of these programs also have a limited library of other nonlinear elements including beam and truss elements. Such software provides the design office with the ability to begin to practically implement nonlinear response history analysis on design projects. However, such software is still limited, and it is expected that it will be some years before design off ices can routinely expect to utilize this technique in the design of complex structures.

5.7.3.1 Member Strength: Nonlinear response history analysis is primarily a deformation based procedure, in which the amount of nonlinear deformation imposed on elements by response to earthquake ground shaking is predicted. As a result, when this analysis method is employed, there is no general need to evaluate the strength demand (forces) imposed on individual elements of the structure. Instead, the adequacy of the individual elements to withstand the imposed deformation demands is directly evaluated, under the requirements of Sec.5.7.4. The exception to this is the requirement to evaluate brittle elements the failure of which could result in structural collapse, for the forces predicted by the analysis. These elements are identified in the *Provisions* through the requirement that they be evaluated for earthquake forces using the special load combinations of Sec. 5.2.7.1. That section requires that forces predicted by elastic analysis be amplified by a factor, Ω_0, to account in an approximate manner for the actual maximum force that can be delivered to the element, considering the inelastic behavior of the structure. Since nonlinear response history analysis does not use a response modification factor, as do elastic analysis approaches, and directly accounts for inelastic structural behavior, there is no need to further increase the forces by this factor. Instead the forces predicted by the analysis are directly used in the evaluation of the elements for adequacy under Sec. 5.2.7.1.

5.7.4 Design Review: The provisions for design using linear methods of analysis including the equivalent lateral force technique of Sec. 5.4 and the modal response spectrum analysis technique of Sec. 5.5, are highly prescriptive. They limit the modeling assumptions that can be employed as well as the minimum strength and stiffness the structure must posses. Further, the methods used in linear analysis have become standardized in practice such that there is unlikely to be substantial difference between the results obtained from different designers using the same technique to analyze the same structure. However, when nonlinear analytical methods are employed to predict the structure's strength and its deformation under load, many of these prescriptive provisions are no longer applicable. Further, as these methods are currently not widely employed by the profession, the standardization that has occurred for linear methods of analysis has not yet been developed for these techniques. As a result analysis has not yet been developed for these techniques, and the designer using such methods must employ a significant amount of independent judgement in developing appropriate analytical models, performing the analysis and interpreting the results to confirm the adequacy of a design. Since relatively minor changes in the assumptions used in performing a nonlinear structural analysis can significantly affect the results obtained from such an analysis, it is imperative that the assumptions used be appropriate. The provisions require that designs employing nonlinear analysis methods be subjected to independent design review in order to provide a level of assurance that the independent judgement applied by the designer when using these methods is appropriate and compatible with those that would be made by other competent practitioners.

5.8 SOIL-STRUCTURE INTERACTION EFFECTS:

5.8.1 General: *Statement of the Problem:* Fundamental to the design requirements presented in Sec. 5.4 and 5.5 is the assumption that the motion experienced by the base of a structure during an earthquake is the same as the "free-field" ground motion, a term that refers to the motion that would occur at the level of the foundation if no structure was present. This assumption implies that the foundation-soil system underlying the structure is rigid and, hence, represents a "fixed-base" condition. Strictly speaking, this assumption never holds in practice. For structures supported on a deformable soil, the foundation motion generally is different from the free-field motion and may include an important rocking *component* in addition to a lateral or translational *component.* The rocking *component*, and soil-structure interaction effects in general, tend to be most significant for laterally stiff structures such as buildings with shear walls, particularly those located on soft soils. For convenience, in what follows the response of a structure supported on a deformable foundation-soil system will be denoted as the "flexible-base" response.

A flexibly supported structure also differs from a rigidly supported structure in that a substantial part of its vibrational energy may be dissipated into the supporting medium by radiation of waves and by hysteretic action in the soil. The importance of the latter factor increases with increasing intensity of ground-shaking. There is, of course, no counterpart of this effect of energy dissipation in a rigidly supported structure.

The effects of soil-structure interaction accounted for in Sec. 5.8 represent the difference in the flexible-base and rigidly supported responses of the structure. This difference depends on the properties of the structure and the supporting medium as well as the characteristics of the free-field ground motion.

The interaction effects accounted for in Sec. 5.8 should not be confused with "site effects," which refer to the fact that the characteristics of the free-field ground motion induced by a dynamic event at a given site are functions of the properties and geological features of the subsurface soil and rock. The interaction effects, on the other hand, refer to the fact that the dynamic response of a structure built on that site depends, in addition, on the interrelationship of the structural characteristics and the properties of the local underlying soil deposits. The site effects are reflected in the values of the seismic coefficients employed in Sec. 5.4 and 5.5 and are accounted for only implicitly in Sec. 5.8.

Possible Approaches to the Problem: Two different approaches may be used to assess the effects of soil-structure interaction. The first involves modifying the stipulated free-field design ground motion, evaluating the response of the given structure to the modified motion of the foundation, and solving simultaneously with additional equations that define the motion of the coupled system, whereas the second involves modifying the dynamic properties of the structure and evaluating the response of the modified structure to the prescribed free-field ground motion (Jennings and Bielak, 1973; Veletsos, 1977). When properly implemented, both approaches lead to equivalent results. However, the second aproach, involving the use of the free-field ground motion, is more convenient for design purposes and provides the basis of the requirements presented in Sec. 5.8.

Characteristics of Interaction: The interaction effects in the approach used here are expressed by an increase in the fundamental natural period of the structure and a change (usually an increase) in its effective damping. The increase in period results from the flexibility of the foundation soil whereas the change in damping results mainly from the effects of energy dissipation in the soil due to radiation and material damping. These statements can be clarified by comparing the responses of rigidly and elastically supported systems subjected to a harmonic excitation of the base. Consider a linear structure of weight W, lateral stiffness k, and coefficient of viscous damping c (shown in Figure C5.8.1-1) and assume that it is supported by a foundation of weight W_o at the surface of a homogeneous, elastic halfspace.

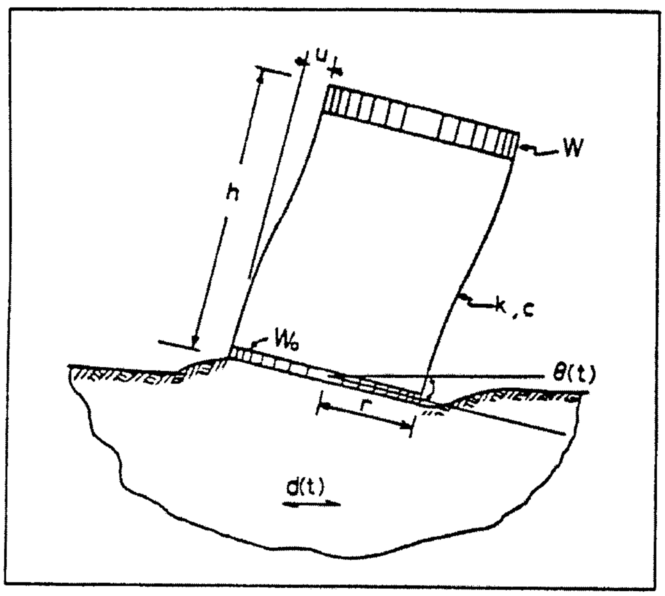


Figure C5.8.1-1 Simple system investigated.

The foundation mat is idealized as a rigid circular plate of negligible thickess bonded to the supporting medium, and the columns of the structure are considered to be weightless and axially inextensible. Both the foundation weight and the weight of the structure are assumed to be uniformly distributed over circular areas of radius r. The base excitation is specified by the free-field motion of the ground surface. This is taken as a horizontally directed, simple harmonic motion with a period T_o and an acceleration amplitude a_m.

The configuration of this system, which has three degrees of freedom when flexibly supported and a single degree of freedom when fixed at the base, is specified by the lateral displacement and rotation of the foundation, y and θ, and by the displacement relative to the base of the top of the structure, u.

The system may be viewed either as the direct model of a one-story structural frame or, more generally, as a model of a multistory, multimode structure that responds as a single-degree-of-freedom system in its fixed-base condition. In the latter case, h must be interpreted as the distance from the base to the centroid of the inertia forces associated with the fundamental mode of vibration of the fixed-base structure and W, k, and c must be interpreted as its generalized or effective weight, stiffness, and damping coefficient, respectively. The relevant expressions for these quantities are given below.

The solid lines in Figures C5.8.1-2 and C5.8.1-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Figure C5.8.1-1. Two different values of h/r and several different values of the relative flexibility parameter for the soil and the structure, ϕ_o, are considered. The latter parameter is defined by the equation $\delta_o = \dfrac{h}{v_s T}$ in which h is the height of the structure as previously indicated, v_s is the velocity of shear wave propagation in the halfspace, and T is the fixed-base natural period of the structure. A value of $\phi = 0$ corresponds to a rigidly supported structure.

The results in Figures C5.8.1-2 and C5.8.1-3 are displayed in a dimensionless form, with the abscissa representing the ratio of the period of the excitation, T_o, to the fixed-base natural period of the system, T, and the ordinate representing the ratio of the amplitude of the actual base shear, V, to the amplitude of the base shear induced in an infinitely stiff, rigidly supported structure. The latter quantity is given by the product ma_m, in which $m = W/g$, g is the acceleration of gravity, and a_m is the acceleration amplitude of the free-field ground motion. The inclined scales on the left represent the deformation amplitude of the superstructure, u, normalized with respect to the displacement amplitude of the free-field ground motion $d_m = \dfrac{a_m T_0^2}{4\pi^2}$.

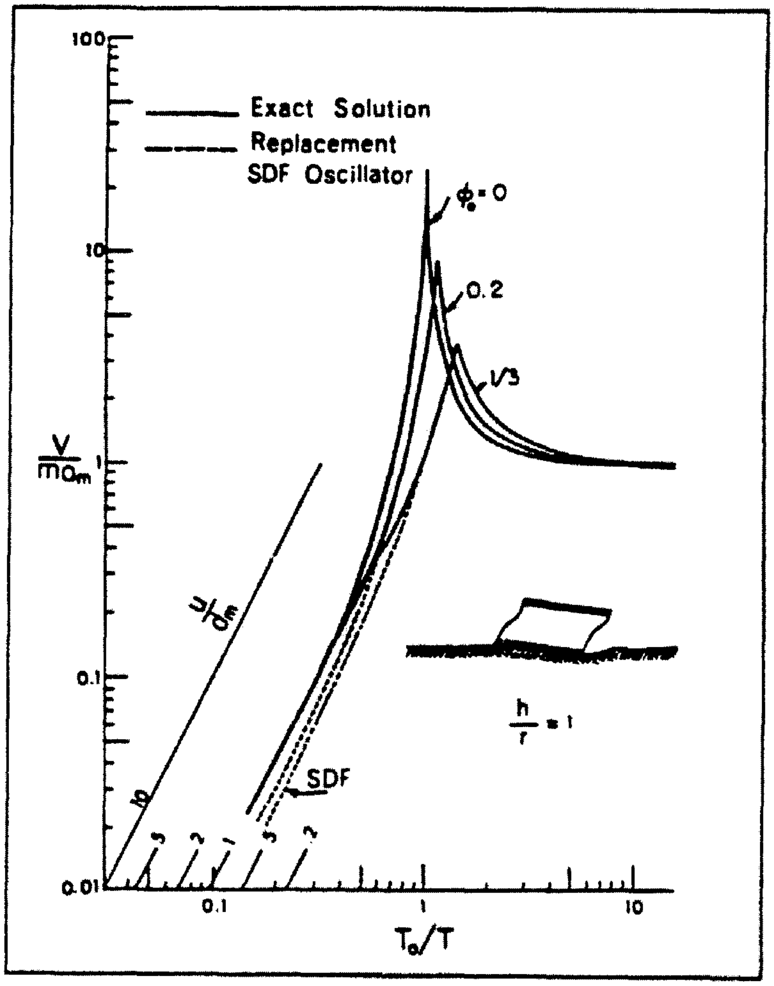


Figure C5.8.1-2 Response spectra for systems with $h/r = 1$ (Veletsos and Meek, 1974).

The damping of the structure in its fixed-base condition, β, is considered to be 2 percent of the critical value, and the additional parameters needed to characterize completely these solutions are identified in Veletsos and Meek (1974), from which these figures have been reproduced.

Comparison of the results presented in these figures reveals that the effects of soil-structure interaction are most strikingly reflected in a shift of the peak of the response spectrum to the right and a change in the magnitude of the peak. These changes, which are particularly prominent for taller structures and more flexible soils (increasing values of ϕ_o), can conveniently be expressed by an increase in the natural period of the system over its fixed-base value and by a change in its damping factor.

Also shown in these figures in dotted lines are response spectra for single-degree-of- freedom (SDF) oscillators, the natural period and damping of which have been adjusted so that the absolute maximum (resonant) value of the base shear and the associated period are in each case identical to those of the actual interacting systems. The base motion for the replacement oscillator is considered to be the same as the free-field ground motion. With the properties of the replacement SDF oscillator determined in this manner, it is important to note that the response spectra for the actual and the replacement systems are in excellent agreement over wide ranges of the exciting period on both sides of the resonant peak.

In the context of Fourier analysis, an earthquake motion may be viewed as the result of superposition of harmonic motions of different periods and amplitudes. Inasmuch as the *components* of the excitation with periods close to the resonant period are likely to be the dominant contributors to the response, the maximum responses of the actual system and of the replacement oscillator can be expected to be in satisfactory agreement for earthquake ground motions as well. This expectation has been confirmed by the results of comprehensive comparative studies (Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975; Jennings and Bielak, 1973).

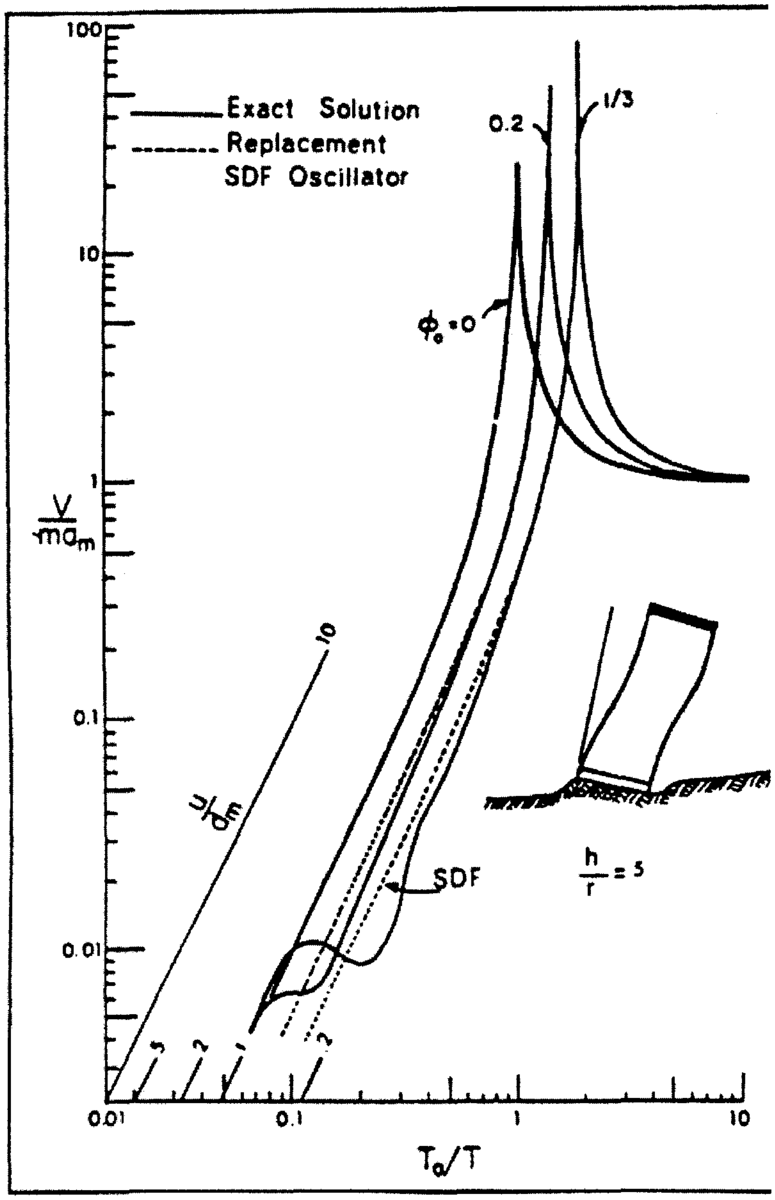


Figure 5.8.1-3 Response spectra for systems with *h/r* = 5 (Veletsos and Meek, 1974).

It follows that, to the degree of approximation involved in the representation of the actual system by the replacement SDF oscillator, the effects of interaction on maximum response may be expressed by an increase in the fundamental natural period of the fixed-base system and by a change in its damping value. In the following sections, the natural period of replacement oscillator is denoted by $\tilde{T}$ and the associated damping factor by $\tilde{\beta}$. These quantities will also be referred to as the effective natural period and the effective damping factor of the interacting system. The relationships between $\tilde{T}$ and T and between $\tilde{\beta}$ and β are considered in Sec. 5.8.2.1.1 and 5.8.2.1.2.

Basis of Provisions and Assumptions: Current knowledge of the effects of soil-structure interactions is derived mainly from studies of systems of the type referred to above in which the foundation is idealized as a rigid mat. For foundations of this type, both surface-supported and embedded structures resting on uniform as well as layered soil deposits have been investigated (Bielak, 1975; Chopra and Gutierrez, 1974; Jennings and Bielak, 1973; Liu and Fagel, 1971; Parmelee et al., 1969; Roesset et al., 1973; Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975). However, the results of such studies may be of limited applicability for foundation systems consisting of individual spread footings or deep foundations (piles or drilled shafts) not interconnected with grade beams or a mat. The requirements presented in Sec. 5.8 for the latter cases represent the best interpretration and judgment of the developers of the requirements regarding the current state of knowledge.

Fundamental to these requirements is the assumption that the structure and the underlying soil are bonded and remain so throughout the period of ground-shaking. It is further assumed that there is no soil instability or large foundation settlements. The design of the foundation in a manner to ensure satisfactory soil performance (e.g., to avoid soil instability and settlement associated with the compaction and liquefaction of loose granular soils), is beyond the scope of Sec. 5.8. Finally, no account is taken of the interaction effects among neighboring structures.

Nature of Interaction Effects: Depending on the characteristics of the structure and the ground motion under consideration, soil-structure interaction may increase, decrease, or have no effect on the magnitudes of the maximum forces induced in the structure itself (Bielak, 1975; Jennings and Bielak, 1973; Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975). However, for the conditions stipulated in the development of the requirements for rigidly supported structures presented in Sec. 5.3 and 5.4, soil-structure interaction will reduce the design values of the base shear and moment from the levels applicable to a rigid-base condition. These forces therefore can be evaluated conservatively without the adjustments recommended in Sec. 5.8.

Because of the influence of foundation rocking, however, the horizontal displacements relative to the base of the elastically supported structure may be larger than those of the corresponding fixed-base structure, and this may increase both the required spacing between structures and the secondary design forces associated with the P-delta effects. Such increases generally are small for frame structures, but can be significant for shear wall structures.

Scope: Two procedures are used to incorporate effects of the soil-structure interaction. The first is an extension of the equivalent lateral force procedure presented in Sec. 5.4 and involves the use of equivalent lateral static forces. The second is an extension of the simplified modal analysis procedure presented in Sec. 5.5. In the latter approach, the earthquake-induced effects are expressed as a linear combination of terms, the number of which is equal to the number of stories involved. Other more complex procedures also may be used, and these are outlined briefly at the end of this commentary on Sec. 5.8. However, it is believed that the more involved procedures are justified only for unusual structures and when the results of the specified simpler approaches have revealed that the interaction effects are indeed of definite consequence in the design.

5.8.2 Equivalent Lateral Force Procedure: This procedure is similar to that used in the older SEAOC recommendations except that it incorporates several improvements (see Sec. 5.4 of this commentary). In effect, the procedure considers the response of the structure in its fundamental mode of vibration and accounts for the contributions of the higher modes implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration. For structures, this assumption has been found to be adequate (Bielak, 1976; Jennings and Bielak, 1973; Veletsos, 1977).

5.8.2.1 Base Shear: With the effects of soil-structure interaction neglected, the base shear is defined by Eq. 5.4.1, $V = C_s W$, in which W is the total dead weight of the structure and of applicable portions of the design live load (as specified in Sec. 5.4.1) and C_s is the dimensionless seismic response coefficient (as defined by Eq. 5.4.1.1-1 and 5.4.1.1-2). This term depends on the seismic zone under consideration, the properties of the site, and the characteristics of the structure itself. The latter characteristics include the rigidly supported fundamental natural period of the structure, T; the associated damping factor, β; and the degree of permissible inelastic deformation. The damping factor does not appear explicitly in Eq. 5.4.1.1-1 and 5.4.1.1-2 because a constant value of $\beta = 0.05$ has been used for all structures for which the interaction effects are negligible. The degree of permissible inelastic action is reflected in the choice of the reduction factor, R. It is convenient to rewrite Eq. 5.4.1 in the form:

$$V = C_s(T,\beta)\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C5.8.2.1-1)}$$

where $\overline{W}$ represents the generalized or effective weight of the structure when vibrating in its fundamental natural mode. The terms in parentheses are used to emphasize the fact that C_s depends upon both T and β. The relationship between $\overline{W}$ and W is given below. The first term on the right side of Eq. C5.8.2.1-1 approximates the contribution of the fundamental mode of vibration whereas the second term approximates the contributions of the higher natural modes. Inasmuch as soil-structure interaction may be considered to affect only the contribution of the fundamental mode and inasmuch as this effect can be expressed by changes in the fundamental natural period and the associated damping of the system, the base shear for the interacting system, $\tilde{V}$, may be stated in a form analogous to Eq. C5.8.2.1-1:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C5.8.2.1-2)}$$

The value of C_s in the first part of this equation should be evaluated for the natural period and damping of the elastically supported system, $\tilde{T}$ and $\tilde{\beta}$, respectively, and the value of C_s in the second term part should be evaluated for the corresponding quantities of the rigidly supported system, T and β.

Before proceeding with the evaluation of the coefficients C_s in Eq. C5.8.2.1-2, it is desirable to rewrite this formula in the same form as Eq. 5.8.2.1-1. Making use of Eq. 5.4.1 and rearranging terms, the following expression for the reduction in the base shear is obtained:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta})\right]\overline{W} \qquad \text{(C5.8.2.1-3)}$$

Within the ranges of natural period and damping that are of interest in studies of structural response, the values of C_s corresponding to two different damping values but the same natural period (e.g., $\tilde{T}$), are related approximately as follows:

$$C_s(\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4} \qquad \text{(C5.8.2.1-4)}$$

This expression, which appears to have been first proposed in Arias and Husid (1962), is in good agreement with the results of studies of earthquake response spectra for systems having different damping values (Newmark et al., 1973).

Substitution of Eq. C5.8.2.1-4 in Eq. C5.8.2.1-3 leads to:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4}\right]\overline{W} \qquad \text{(C5.8.2.1-5)}$$

where both values of C_s are now for the damping factor of the rigidly supported system and may be evaluated from Eq. 5.4.1.1-1 and 5.4.1.1-2. If the terms corresponding to the periods T and $\tilde{T}$ are denoted more simply as C_s and $\tilde{C}_s$, respectively, and if the damping factor β is taken as 0.05, Eq. C5.8.2.1-5 reduces to Eq. 5.8.2.1-2.

Note that $\tilde{C}_s$ in Eq. 5.8.2.1-2 is smaller than or equal to C_s because Eq. 5.4.1 is a nonincreasing function of the natural period and $\tilde{T}$ is greater than or equal to T. Furthermore, since the minimum value of $\tilde{\beta}$ is taken as $\tilde{\beta} = \beta = 0.05$ (see statement following Eq. 5.8.2.1.2-1), the shear reduction ΔV is a non-negative quantity. It follows that the design value of the base shear for the elastically supported structure cannot be greater than that for the associated rigid -base structure.

The effective weight of the structure, $\overline{W}$, is defined by Eq. 5.5.4-2 (Sec. 5.5), in which ϕ_{im} should be interpreted as the displacement amplitude of the i^{th} floor when the structure is vibrating in its fixed-base fundamental natural mode. It should be clear that the ratio $\overline{W} / W$ depends on the detailed characteristics of the structure. A constant value of $\overline{W} = 0.7\ W$ is recommended in the interest of simplicity and because it is a good approximation for typical structures. As an example, it is noted that for a tall structure for which the weight is uniformly distributed along the height and for which the fundamental natural mode increases linearly from the base to the top, the exact value of $\overline{W}$ = 0.75 W. Naturally, when the full weight of the structure is concentrated at a single level, $\overline{W}$ should be taken equal to W.

The maximum permissible reduction in base shear due to the effects of soil-structure interaction is set at 30 percent of the value calculated for a rigid-base condition. It is expected, however, that this limit will control only infrequently and that the calculated reduction, in most cases, will be less.

5.8.2.1.1 Effective Building Period: Equation 5.8.2.1.1-1 for the effective natural period of the elastically supported structure, $\tilde{T}$, is determined from analyses in which the superstructure is presumed to respond in its fixed-base fundamental mode and the foundation weight is considered to be negligible in comparison to the weight of the superstructure (Jennings and Bielak, 1973; Veletsos and Meek, 1974). The first term under the radical represents the period of the fixed-base structure. The first portion of the second term represents the contribution to $\tilde{T}$ of the translational flexibility of the foundation, and the last portion represents the contribution of the corresponding rocking flexibility. The quantities $\bar{k}$ and $\bar{h}$ represent, respectively, the effective stiffness and effective height of the structure, and K_y and K_θ represent the translational and rocking stiffnesses of the foundation.

Equation 5.8.2.1.1-2 for the structural stiffness, $\bar{k}$, is deduced from the well known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{\left(\frac{1}{g}\right)\left(\frac{\overline{W}}{\bar{k}}\right)} \qquad \text{(C5.8.2.1.1-1)}$$

The effective height, $\bar{h}$, is defined by Eq. 5.8.3.1-2, in which ϕ_{il} has the same meaning as the quantity ϕ_{im} in Eq. 5.5.4.-2 when $m = 1$. In the interest of simplicity and consistency with the approximation used in the definition of $\overline{W}$, however, a constant value of $\bar{h} = 0.7h_n$ is recommended where h_n is the total height of the structure. This value represents a good approximation for typical structures. As an example, it is noted that for tall structures for which the fundamental natural mode increases linearly with height, the exact value of $\bar{h}$ is $2/3h_n$. Naturally, when the gravity load of the structure is effectively concentrated at a single level, h_n must be taken as equal to the distance from the base to the level of weight concentration.

Foundation stiffnesses depend on the geometry of the foundation-soil contact area, the properties of the soil beneath the foundation, and the characteristics of the foundation motion. Most of the available information on this subject is derived from analytical studies of the response of harmonically excited rigid circular foundations, and it is desirable to begin with a brief review of these results.

For circular mat foundations supported at the surface of a homogeneous halfspace, stiffnesses K_y and K_θ are given by:

$$K_y = \left[\frac{8\alpha_y}{2 - \nu}\right] Gr \qquad \text{(C5.8.2.1.1-2)}$$

and

$$K_\theta = \left[\frac{8\alpha_\theta}{3(1-\nu)} \right] Gr^3 \qquad \text{(C5.8.2.1.1-3)}$$

where r is the radius of the foundation; G is the shear modulus of the halfspace; ν is its Poisson's ratio; and α_y and α_θ are dimensionless coefficients that depend on the period of the excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco, 1974; Veletsos and Verbic, 1974; Veletsos and Wei, 1971). The shear modulus is related to the shear wave velocity, v_s, by the formula:

$$G = \frac{\gamma v_s^2}{g} \qquad \text{(C5.8.2.1.1-4)}$$

in which γ is the unit weight of the material. The values of G, v_s, and ν should be interpreted as average values for the region of the soil that is affected by the forces acting on the foundation and should correspond to the conditions developed during the design earthquake. The evaluation of these quantities is considered further in subsequent sections. For statically loaded foundations, the stiffness coefficients α_y and α_θ are unity, and Eq. C5.8.2.1.1-2 and 5.8.2.1.1-3 reduce to:

$$K_y = \frac{8Gr}{2-\nu} \qquad \text{(C5.8.2.1.1-5)}$$

and

$$K_\theta = \frac{8Gr^3}{3(1-\nu)} \qquad \text{(C5.8.2.1.1-6)}$$

Studies of the interaction effects in structure-soil systems have shown that, within the ranges of parameters of interest for structures subjected to earthquakes, the results are insensitive to the period-dependency of α_y and that it is sufficiently accurate for practical purposes to use the static stiffness K_y, defined by Eq. C5.8.2.1.1-5. However, the dynamic modifier for rocking α_θ can significantly affect the response of building structures. In the absence of more detailed analyses, for ordinary building structures with an embedment ratio $d/r < 0.5$, the factor α_θ can be estimated as follows:

r/v_sT	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

where d equals depth of embedment and r can be taken as r_m defined in Eq. 5.8.2.1.2-3.

The above values were derived from the solution for α_θ by Veletsos and Verbic (1973). In this solution α_θ is a function of $\tilde{T}$. To relate α_θ to T, a correction for period lengthening ($\tilde{T}/T$) was made assuming $\bar{h}/r \sim 0.5$ to 1.0 and Poisson's ratio $\nu = 0.4$.

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas:

$$K_y = \left[\frac{8Gr}{2 - \nu}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right] \tag{C5.8.2.1.1-7}$$

and

$$K_\theta = \left[\frac{8Gr^3\alpha_\theta}{3(1 - \nu)}\right]\left[1 + 2\left(\frac{d}{r}\right)\right] \tag{C5.8.2.1.1-8}$$

in which d is the depth of embedment. These formulas are based on finite element solutions (Kausel, 1974).

Both analyses and available test data (Erden, 1974) indicate that the effects of foundation embedment are sensitive to the condition of the backfill and that judgment must be exercised in using Eq. C5.8.2.1.1-7 and C5.8.2.1.1-8. For example, if a structure is embedded in such a way that there is no positive contact between the soil and the walls of the structure, or when any existing contact cannot reasonably be expected to remain effective during the stipulated design ground motion, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d in Eq. C5.8.2.1.1-7 and C5.8.2.1.1-8 should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake.

The formulas for K_y and K_θ presented above are strictly valid only for foundations supported on reasonably uniform soil deposits. When the foundation rests on a surface stratum of soil underlain by a stiffer deposit with a shear wave velocity (v_s) more than twice that of the surface layer (Wallace et al., 1999), K_y and K_θ may be determined from the following two generalized formulas in which G is the shear modulus of the soft soil and D_s is the total depth of the stratum. First, using Eq. C5.8.2.1.1-7:

$$K_y \approx \left[\frac{8Gr}{2 - \nu}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{2}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + \left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \qquad \text{(C5.8.2.1.1-9)}$$

Second, using Eq. C5.8.2.1.1-8:

$$K_\theta \approx \left[\frac{8Gr^3\alpha_\theta}{3(1 - \nu)}\right]\left[1 + 2\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{6}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + 0.7\left(\frac{d}{D_s}\right)\right] \qquad \text{(C5.8.2.1.1-10)}$$

These formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al.,1977; Kausel and Roesset, 1975) and apply for $r/D_s < 0.5$ and $d/r < 1$.

The information for circular foundations presented above may be applied to mat foundations of arbitrary shapes provided the following changes are made:

1. The radius r in the expressions for K_y is replaced by r_a (Eq. 5.8.2.1.1-5), which represents the radius of a disk that has the area, A_o, of the actual foundation.

2. The radius r in the expressions for K_θ is replaced by r_m (Eq. 5.8.2.1.1-6), which represents the radius of a disk that has the moment of inertia, I_o, of the actual foundation.

For footing foundations, stiffnesses K_y and K_θ are computed by summing the contributions of the individual footings. If it is assumed that the foundation behaves as a rigid body and that the individual footings are widely spaced so that they act as independent units, the following formulas are obtained:

$$K_y = \Sigma k_{yi} \qquad \text{(C5.8.2.1.1-11)}$$

and

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i} \qquad \text{(C5.8.2.1.1-12)}$$

The quantity k_{yi} represents the horizontal stiffness of the i^{th} footing; k_{xi} and $k_{\theta i}$ represent, respectively, the corresponding vertical and rocking stiffnesses; and y_i represents the normal distance from the centroid of the i^{th} footing to the rocking axis of the foundation. The summations are considered to extend over all footings. The contribution to K_θ of the rocking stiffnesses of the individual footings, $k_{\theta i}$, generally is small and may be neglected.

The stiffnesses k_{yi}, k_{xi}, and $k_{\theta i}$ are defined by the formulas:

$$k_{yi} = \left(\frac{8G_i r_{ai}}{2 - \nu}\right)\left(1 + \frac{2}{3}\frac{d}{r}\right) \quad \text{(C5.8.2.1.1-13)}$$

$$k_{xi} = \left(\frac{4G_i r_{ai}}{1 - \nu}\right)\left(1 + 0.4\frac{d}{r}\right) \quad \text{(C5.8.2.1.1-14)}$$

and

$$k_{\theta i} = \left[\frac{8G_i r_{mi}^3}{3(1 - \nu)}\right]\left[1 + 2\frac{d}{r}\right] \quad \text{(C5.8.2.1.1-15)}$$

in which d_i is the depth of effective embedment for the i^{th} footing; G_i is the shear modulus of the soil beneath the i^{th} footing; $r_{ai} = \sqrt{A_{oi}/\pi}$ is the radius of a circular footing that has the area of the i^{th} footing, A_{oi}; and r_{mi} equals $\sqrt[4]{4I_{oi}/\pi}$ the radius of a circular footing, the moment of inertia of which about a horizontal centroidal axis is equal to that of the i^{th} footing, I_{oi}, in the direction in which the response is being evaluated.

For surface-supported footings and for embedded footings for which the side wall contact with the soil cannot be considered to be effective during the stipulated design ground motion, d_i in these formulas should be taken as zero. Furthermore, the values of G_i should be consistent with the stress levels expected under the footings and should be evaluated with due regard for the effects of the dead loads involved. This matter is considered further in subsequent sections. For closely spaced footings, consideration of the coupling effects among footings will reduce the computed value of the overall foundation stiffness. This reduction will, in turn, increase the fundamental natural period of the system, $\tilde{T}$, and increase the value of ΔV, the amount by which the base shear is reduced due to soil-structure interaction. It follows that the use of Eq. C5.8.2.1.1-11 and 5.8.2.1.1-12 will err on the conservative side in this case. The degree of conservatism involved, however, will partly be compensated by the presence of a basement slab that, even when it is not tied to the structural frame, will increase the overall stiffness of the foundation.

The values of K_y and K_θ for pile foundations can be computed in a manner analogous to that described in the preceding section by evaluating the horizontal, vertical, and rocking stiffnesses of the individual piles, k_{yi}, k_{xi} and $k_{\theta i}$, and by combining these stiffnesses in accordance with Eq. C5.8.2.1.1-11 and 5.8.2.1.1-12.

The individual pile stiffnesses may be determined from field tests or analytically by treating each pile as a beam on an elastic subgrade. Numerous formulas are available in the literature (Tomlinson,

1994) that express these stiffnesses in terms of the modulus of the subgrade reaction and the properties of the pile itself. These stiffnesses sometimes are expressed in terms of the stiffness of an equivalent freestanding cantilever, the physical properties and cross-sectional dimensions of which are the same as those of the actual pile but the length of which is adjusted appropriately. The effective lengths of the equivalent cantilevers for horizontal motion and for rocking or bending motion are slightly different but are often assumed to be equal. On the other hand, the effective length in vertical motion is generally considerably greater.

The soil properties of interest are the shear modulus, G, or the associated shear wave velocity, v_s; the unit weight, γ; and Poisson's ratio, ν. These quantities are likely to vary from point to point of a construction site, and it is necessary to use average values for the soil region that is affected by the forces acting on the foundation. The depth of significant influence is a function of the dimensions of the foundation base and of the direction of the motion involved. The effective depth may be considered to extend to about $0.75r_a$ below the foundation base for horizontal motions, $2r_a$ for vertical motions, and to about $0.75r_m$ for rocking motion. For mat foundations, the effective depth is related to the total plan dimensions of the mat whereas for structures supported on widely spaced spread footings, it is related to the dimensions of the individual footings. For closely spaced footings, the effective depth may be determined by superposition of the "pressure bulbs" induced by the forces acting on the individual footings.

Since the stress-strain relations for soils are nonlinear, the values of G and v_s also are functions of the strain levels involved. In the formulas presented above, G should be interpreted as the secant shear modulus corresponding to the significant strain level in the affected region of the foundation soil. The approximate relationship of this modulus to the modulus G_o corresponding to small amplitude strains (of the order of 10^{-3} percent or less) is given in Table 5.8.2.1.1. The backgrounds of this relationship and of the corresponding relationship for v_s/v_{so} are identified below.

The low amplitude value of the shear modulus, G_o, can most conveniently be determined from the associated value of the shear wave velocity, v_{so}, by use of Eq. C5.8.2.1.1-4. The latter value may be determined approximately from empirical relations or more accurately by means of field tests or laboratory tests.

The quantities G_o and v_{so} depend on a large number of factors (Hardin, 1978), the most important of which are the void ratio, e, and the average confining pressure, $\overline{\sigma}_o$. The value of the latter pressure at

$$\overline{\sigma}_o = \overline{\sigma}_{os} + \overline{\sigma}_{ob} \qquad \text{(C5.8.2.1.1-16)}$$

a given depth beneath a particular foundation may be expressed as the sum of two terms as follows: in which $\overline{\sigma}_{os}$ represents the contribution of the weight of the soil and $\overline{\sigma}_{ob}$ represents the contribution of the superimposed weight of the structure and foundation. The first term is defined by the formula:

$$\overline{\sigma}_{os} = \left(\frac{1 + 2K_o}{3} \right) \gamma' x \qquad \text{(C5.8.2.1.1-17)}$$

in which x is the depth of the soil below the ground surface, γ' is the average effective unit weight of the soil to the depth under consideration, and K_o is the coefficient of horizontal earth pressure at rest. For sands and gravel, K_o has a value of 0.5 to 0.6 whereas for soft clays, $K_o \simeq 1.0$. The pressures $\overline{\sigma_{ob}}$ developed by the weight of the structure can be estimated from the theory of elasticity (Poulos and Davis, 1974). In contrast to $\overline{\sigma_{os}}$ which increases linearly with depth, the pressures $\overline{\sigma_{ob}}$ decrease with depth. As already noted, the value of v_{so} should correspond to the average value of $\overline{\sigma_o}$ in the region of the soil that is affected by the forces acting on the foundation.

For clean sands and gravels having $e < 0.80$, the low-amplitude shear wave velocity can be calculated

$$v_{so} = c_1(2.17 - e)(\overline{\sigma})^{0.25} \tag{C5.8.2.1.1-18}$$

approximately from the formula:

in which c_1 equals 78.2 when $\overline{\sigma}$ is in lb/ft^2 and v_{so} is in ft/sec; c_1 equals 160.4 when $\overline{\sigma}$ is in kg/cm^2 and v_{so} is in m/sec; and c_1 equals 51.0 when $\overline{\sigma}$ is in kN/m^2 and v_{so} is in m/sec.

$$v_{so} = c_2(2.97 - e)(\overline{\sigma})^{0.25} \tag{C5.8.2.1.1-19}$$

For angular-grained cohesionless soils ($e > 0.6$), the following empirical equation may be used:

in which c_2 equals 53.2 when $\overline{\sigma}$ is in lb/ft^2 and v_{so} is in ft/sec; c_2 equals 109.7 when $\overline{\sigma}$ is in kg/cm^2 and v_{so} is in m/sec; and c_2 equals 34.9 when $\overline{\sigma}$ is in kN/m^2 and v_{so} is in m/sec.

Equation C5.9.2.1.1-19 also may be used to obtain a first-order estimate of v_{so} for normally consolidated cohesive soils. A crude estimate of the shear modulus, G_o, for such soils may also be obtained from the relationship:

$$G_o = 1{,}000 S_u. \tag{C5.8.2.1.1-20}$$

in which S_u is the shearing strength of the soil as developed in an unconfined compression test. The coefficient 1,000 represents a typical value, which varied from 250 to about 2,500 for tests on different soils (Hara et al., 1974; Hardin and Drnevich, 1975).

These empirical relations may be used to obtain preliminary, order-of-magnitude estimates. For more accurate evaluations, field measurements of v_{so} should be made. Field evaluations of the variations of v_{so} throughout the construction site can be carried out by standard seismic refraction methods, the downhole or cross-hole methods, suspension logging, or spectral analysis with surface waves. Kramer (1996) provides an overview of these testing procedures. The disadvantage of these methods are that v_{so} is determined only for the stress conditions existing at the time of the test (usually $\overline{\sigma_{so}}$). The effect of the changes in the stress conditions caused by construction must be considered by use of

Eq. C5.8.2.1.1-17 and Eq. C5.8.2.1.1-18 and C5.8.2.1.1-19 to adjust the field measurement of v_{so} to correspond to the prototype situations. The influence of large-amplitude shearing strains may be evaluated from laboratory tests or approximated through the use of Table 5.8.2.1.1. This matter is considered further in the next two sections.

An increase in the shearing strain amplitude is associated with a reduction in the secant shear modulus, G, and the corresponding value of v_s. Extensive laboratory tests (for example, Vucetic and Dobry, 1991; Seed et al., 1984) have established the magnitudes of the reductions in v_s for both sands and clays as the shearing strain amplitude increases.

The results of such tests form the basis for the information presented in Table 5.8.2.1.1. For each severity of anticipated ground-shaking, represented by the effective peak acceleration coefficients A_a and A_v, a representative value of shearing strain amplitude was developed. A conservative value of v_s/v_{so} that is appropriate to that strain amplitude then was established. It should be emphasized that the values in Table 5.8.2.1.1 are first order approximations. More precise evaluations would require the use of material-specific shear modulus reduction curves and studies of wave propagation for the site to determine the magnitude of the soil strains induced.

It is satisfactory to assume Poisson's ratio for soils as: $\nu = 0.33$ for clean sands and gravels, $\nu = 0.40$ for stiff clays and cohesive soils, and $\nu = 0.45$ for soft clays. The use of an average value of $\nu = 0.4$ also will be adequate for practical purposes.

Regarding an alternative approach, note that Eq. 5.8.2.1.1-3 for the period $\tilde{T}$ of structures supported on mat foundations was deduced from Eq. 5.8.2.1.1-1 by making use of Eq. C5.8.2.1.1-5 and C5.8.2.1.1-6, with Poisson's ratio taken as $\nu = 0.4$ and with the radius r interpreted as r_a in Eq. C5.8.2.1.1-5 and as r_m in Eq. C5.8.2.1.1-6. For a nearly square foundation, for which $r_a \approx r_m \approx r$, Eq. 5.8.2.1.1-3 reduces to:

$$\tilde{T} = T\sqrt{1 + 25\alpha\left(\frac{r\bar{h}}{v_s^2 T^2}\right)\left[1 + \left(\frac{1.12\bar{h}^2}{\alpha_\theta r^2}\right)\right]} \qquad \text{(C5.8.2.1.1-21)}$$

The value of the relative weight parameter, α, is likely to be in the neighborhood of 0.15 for typical structures.

5.8.2.1.2 Effective Damping: Equation 5.8.2.1.2-1 for the overall damping factor of the elastically supported structure, $\tilde{\beta}$, was determined from analyses of the harmonic response at resonance of simple systems of the type considered in Figures C5.8.1-2 and 5.8.1-3. The result is an expression of the form (Bielak, 1975; Veletsos and Nair, 1975):

$$\tilde{\beta} = \beta_o + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \qquad \text{(C5.8.2.1.2-1)}$$

in which β_o represents the contribution of the foundation damping, considered in greater detail in the following paragraphs, and the second term represents the contribution of the structural damping. The latter damping is assumed to be of the viscous type. Equation C5.8.2.1.2-1 corresponds to the value of $\beta = 0.05$ used in the development of the response spectra for rigidly supported systems employed in Sec. 5.4.

The foundation damping factor, β_o, incorporates the effects of energy dissipation in the soil due to the following sources: the radiation of waves away from the foundation, known as radiation or geometric damping, and the hysteretic or inelastic action in the soil, also known as soil material damping. This factor depends on the geometry of the foundation-soil contact area and on the properties of the structure and the underlying soil deposits.

For mat foundations of circular plan that are supported at the surface of reasonably uniform soils deposits, the three most important parameters which affect the value of β_o are: the ratio $\tilde{T}/T$ of the fundamental natural periods of the elastically supported and the fixed-base structures, the ratio $\bar{h}/r$ of the effective height of the structure to the radius of the foundation, and the damping capacity of the soil. The latter capacity is measured by the dimensionless ratio $\Delta W_s/W_s$, in which ΔW_s is the area of the hysteresis loop in the stress-strain diagram for a soil specimen undergoing harmonic shearing deformation and W_s is the strain energy stored in a linearly elastic material subjected to the same maximum stress and strain (i.e., the area of the triangle in the stress-strain diagram between the origin and the point of the maximum induced stress and strain). This ratio is a function of the magnitude of the imposed peak strain, increasing with increasing intensity of excitation or level of strain.

The variation of β_o with $\tilde{T}/T$ and $\bar{h}/r$ is given in Figure 5.8.2.1.2 for two levels of excitation. The dashed lines, which are recommended for values of the design earthquake spectral response acceleration at short periods, S_{DS}, equal to or less than 0.25, correspond to a value of $\Delta W_s/W_s \simeq 0.3$, whereas the solid lines, which are recommended for S_{DS} values equal to or greater than 0.20, correspond to a value of $\Delta W_s/W_s \simeq 1$. These curves are based on the results of extensive parametric studies (Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975) and represent average values. For the ranges of parameters that are of interest in practice, however, the dispersion of the results is small.

For mat foundations of arbitrary shape, the quantity r in Figure 5.8.2.1.2 should be interpreted as a characteristic length that is related to the length of the foundation, L_o, in the direction in which the structure is being analyzed. For short, squatty structures for which $\bar{h}/L_o \leq 0.5$, the overall damping of the structure-foundation system is dominated by the translational action of the foundation, and it is reasonable to interpret r as r_a, the radius of a disk that has the same area as that of the actual foundation (see Eq. 5.8.2.1.1-5). On the other hand, for structures with $\bar{h}/L_o \geq 1$, the interaction effects are dominated by the rocking motion of the foundation, and it is reasonable to define r as the radius r_m of a disk whose static moment of inertia about a horizontal centroidal axis is the same as that of the actual foundation normal to the direction in which the structure is being analyzed (see Eq. 5.8.2.1.1-6).

Subject to the qualifications noted in the following section, the curves in Figure 5.8.2.1.2 also may be used for embedded mat foundations and for foundations involving spread footings or piles. In the latter cases, the quantities A_o and I_o in the expressions for the characteristic foundation length, r, should be interpreted as the area and the moment of inertia of the load-carrying foundation.

In the evaluation of the overall damping of the structure-foundation system, no distinction has been made between surface-supported foundations and embedded foundations. Since the effect of embedment is to increase the damping capacity of the foundation (Bielak, 1975; Novak, 1974; Novak and Beredugo, 1972) and since such an increase is associated with a reduction in the magnitude of the forces induced in the structure, the use of the recommended requirements for embedded structures will err on the conservative side.

There is one additional source of conservatism in the application of the recommended requirements to structures with embedded foundations. It results from the assumption that the free-field ground motion at the foundation level is independent of the depth of foundation embedment. Actually, there is evidence to the effect that the severity of the free-field excitation decreases with depth (Seed et al., 1977). This reduction is ignored both in Sec. 5.8 and in the requirements for rigidly supported structures presented in Sec. 5.4 and 5.5.

Equations 5.8.2.1.2-1 and C5.8.2.1.2-2, in combination with the information presented in Figure 5.8.2.1.2, may lead to damping factors for the structure-soil system, $\tilde{\beta}$, that are smaller than the structural damping factor, β. However, since the representative value of $\beta = 0.05$ used in the development of the design requirements for rigidly supported structures is based on the results of tests on actual structures, it reflects the damping of the full structure-soil system, not merely of the *component* contributed by the superstructure. Thus, the value of $\tilde{\beta}$ determined from Eq. 5.8.2.1.2-1 should never be taken less than β, and a low bound of $\tilde{\beta} = \beta = 0.05$ has been imposed. The use of values of $\tilde{\beta} > \beta$ is justified by the fact that the experimental values correspond to extremely small amplitude motions and do not reflect the effects of the higher soil damping capacities corresponding to the large soil strain levels associated with the design ground motions. The effects of the higher soil damping capacities are appropriately reflected in the values of β_o presented in Figure 5.8.2.1.5.

There are, however, some exceptions. For foundations involving a soft soil stratum of reasonably uniform properties underlain by a much stiffer, rock-like material with an abrupt increase in stiffness, the radiation damping effects are practically negligible when the natural period of vibration of the stratum in shear,

$$T_s = \frac{4D_s}{v_s} \qquad \text{(C5.8.2.1.2-2)}$$

is smaller than the natural period of the flexibly supported structure, $\tilde{T}$. The quantity D_s in this formula represents the depth of the stratum. It follows that the values of β_o presented in Figure 5.8.2.1.2 are applicable only when:

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \geq 1 \qquad \text{(C5.8.2.1.2-3)}$$

for

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \leq 1 \qquad \text{(C5.8.2.1.2-4)}$$

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} < 1 \qquad \text{(C5.8.2.1.2-4)}$$

the effective value of the foundation damping factor, β_o', is less than β_o, and it is approximated by the second degree parabola defined by Eq. 5.8.2.1.2-4.

For $T_s/\tilde{T} = 1$, Eq. 5.8.2.1.2-4 leads to $\beta_o' = \beta_o$ whereas for $T_s/\tilde{T} = 0$, it leads to $\beta_o' = 0$, a value that clearly does not provide for the effects of material soil damping. It may be expected, therefore, that the computed values of β_o' corresponding to small values of $T_s/\tilde{T}$ will be conservative. The conservatism involved, however, is partly compensated by the requirement that $\tilde{\beta}$ be no less than $\tilde{\beta} = \beta = 0.05$.

5.8.2.2 and 5.8.2.3 Vertical Distribution of Seismic Forces and Other Effects: The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are generally different. However, the differences are inconsequential for practical purposes, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the requirements for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures. The above procedure is applicable to planar structures and, with some extension, to three-dimensional structures. Methods exist for incorporating two- and three-dimensional *P*-delta effects into computer analyses that do not explicable include such effects (Rutenburg, 1985). Many programs explicitly include *P*-delta effects. A mathematical description of the method employed by several popular programs is given by Wilson and Habibullah (1987).

The *P*-delta procedure cited above effectively checks the static stability of a structure based on its initial stiffness. Since the inception of this procedure in the ATC 3-06 document, however, there has been some debate regarding its accuracy. This debate reflects the intuitive notion that a structure's secant stiffness would more accurately represent inelastic *P*-delta effects. Due to the additional uncertainty of the effect of dynamic response on *P*-delta behavior and on the (apparent) observation that instability-related failures rarely occur in real structures, the *P*-delta requirements as originally written have remained unchanged until now.

There is increasing evidence, however, that the use of inelastic stiffness in determining theoretical *P*-delta response is unconservative. Based on a study carried out by Bernal (1987), it can be argued that *P*-delta amplifiers should be based on secant stiffness. In other words, the C_d term in Eq. 5.4.6.2.-1 of the *Provisions* should be deleted. Since Bernal's study was based on the inelastic dynamic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties exist in the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

Another problem with accepting a *P*-delta procedure based on secant stiffness is that current design forces would be greatly increased. For example, consider an ordinary moment frame of steel with a C_d of 4.0 and an elastic stability coefficient, θ, of 0.15. The amplifier for this structure would be 1.0/0.85 = 1.18 according to the current requirements. If the *P*-delta effects were based on secant stiffness, however, the stability coefficient would increase to 0.60 and the amplifier would become 1.0/0.4 = 5.50. (Note that the 0.9 in the numerator of the amplifier equation in the 1988 Edition of the *Provisions* has been dropped for this comparison.) From this example, it can be seen that there could be an extreme impact on the requirements if a change was implemented that incorporated *P*-delta amplifiers based on static secant stiffness response.

Nevertheless, there must be some justification for retaining the *P*-delta amplifier as based on elastic stiffness. This justification is the apparent lack of stability-related failures. The reasons for the lack of observed failures are, at a minimum, twofold:

1. Many structures display an overstrength well above the strength implied by code-level design forces (see Figure 5.8.1). This overstrength likely protects structures from stability-related failures.

5. The likelihood of a stability failure decreases with the increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most structures designed for extreme ground motion is significantly greater than the stiffness of the same structure deigned for lower intensity shaking or for wind. Since damaging low-intensity earthquakes are somewhat rare, there would be little observable damage.

Due to the lack of stability-related failures, therefore, the 1991 Edition of the *Provisions* regarding *P*-delta amplifiers has remained unchanged from the 1988 Edition with the exception that the 0.90 factor in the numerator of the amplifier has been deleted. This factor originally was used to create a transition from cases where *P*-delta effects need not be considered ($\theta > 1.0$, amplifier > 1.0).

Aside from the amplifier, however, the 1991 Edition of the *Provisions* added a new requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$ where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 8 though 14 of the *Provisions*.

The purpose of this new provision is to protect structures from the possibility of stability-related failures triggered by post-earthquake residual deformation. The danger of such failures is real and may not be eliminated by apparently available overstrength. This is particularly true of structures designed in for regions of lower seismicity.

The computation of θ_{max}, which in turn is based on βC_d, requires the computation of story strength supply and story strength demand. Story strength demand is simply the seismic design shear for the story under consideration. The story strength supply may be computed as the shear in the story that occurs simultaneously with the attainment of the development of first significant yield of the overall structure. To compute first significant yield, the structure should be loaded with a seismic force pattern similar to that used to compute seismic story strength demand. A simple and conservative procedure is to compute the ratio of demand to strength for each member of the seismic-force-resisting system in a particular story and then use the largest such ratio as β. For a structure otherwise in conformance with the *Provisions*, $\beta = 1.0$ is obviously conservative.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of adding stiffness for drift control, of code-required wind resistance, or simply of a feature of other aspects of the design.

5.8.3 Modal Analysis Procedure: Studies of the dynamic response of elastically supported multi--degree-of-freedom systems (Bielak, 1976; Chopra and Gutierrez, 1974; Veletsos, 1977) reveal that, within the ranges of parameters that are of interest in the design of structures subjected to earthquakes, soil-structure interaction affects substantially only the response *component* contributed by the fundamental mode of vibration of the superstructure. In this section, the interaction effects are considered only in evaluating the contribution of the fundamental structural mode. The contributions of the higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined, as for rigidly supported structures, by taking the square root of the sum of the squares of the maximum modal contributions.

The interaction effects associated with the response in the fundamental structural mode are determined in a manner analogous to that used in the analysis of the equivalent lateral force method, except that the effective weight and effective height of the structure are computed so as to correspond exactly to those of the fundamental natural mode of the fixed-base structure. More specifically, $\overline{W}$ is computed from:

$$\overline{W} = \overline{W}_1 = \frac{\left(\Sigma w_i \phi_{il}\right)^2}{\Sigma w_i \phi_{il}^2} \tag{C5.8.3}$$

which is the same as Eq. 5.5.4-2, and $\bar{h}$ is computed from Eq. 5.8.3.1-2. The quantity ϕ_{il} in these formulas represents the displacement amplitude of the i^{th} floor level when the structure is vibrating in its fixed-base fundamental natural mode. The structural stiffness, $\bar{k}$, is obtained from Eq. 5.8.2.1.1-2 by taking $\overline{W} = \overline{W}_l$ and using for T the fundamental natural period of the fixed-base structure, T_l. The fundamental natural period of the interacting system, $\tilde{T}_l$, is then computed from Eq. 5.8.2.1.1-1 (or Eq. 5.8.2.2.1.1-3 when applicable) by taking $T = T_l$. The effective damping in the first mode, β, is determined from Eq. 5.8.2.1.2-1 (and Eq. 5.8.2.1.2-4 when applicable) in combination with the information given in Figure 5.8.2.1.2. The quantity $\bar{h}$ in the latter figure is computed from Eq. 5.8.3.1-2.

With the values of $\tilde{T}_l$ and $\tilde{\beta}_l$ established, the reduction in the base shear for the first mode, ΔV_l, is computed from Eq. 5.8.2.1-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_l and $\tilde{T}_l$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_l$; and $\overline{W}$ should be determined from Eq. C5.8.3.

The sections on lateral forces, shears, overturning moments, and displacements follow directly from what has already been noted in this and the preceding sections and need no elaboration. It may only be pointed out that the first term within the brackets on the right side of Eq. 5.8.3.2-1 represents the contribution of the foundation rotation.

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

The effects of torsion about a vertical axis should be evaluated in accordance with the requirements of Sec. 5.4.4 and the *P*-delta effects should be evaluated in accordance with the requirements of Sec. 5.4.6.2, using the story shears and drifts determined in Sec. 5.8.3.2.

Other Methods of Considering the Effects of Soil Structure Interaction: The procedures proposed in the preceding sections for incorporating the effects of soil-structure interaction provide sufficient flexibility and accuracy for practical applications. Only for unusual structures and only when the requirements indicate that the interaction effects are of definite consequence in design, would the use of more elaborate procedures be justified. Some of the possible refinements, listed in order of more or less increasing complexity, are:

1. Improve the estimates of the static stiffnesses of the foundation, K_y and K_θ, and of the foundation damping factor, β_o, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil. Solutions may be obtained in some cases with analytical or semi-analytical formulations and in others by application of finite difference or finite element techniques. A concise review of available analytical formulations is provided in Gazetas (1991). It should be noted, however, that these solutions involve approximations of their own that may offset, at least in part, the apparent increase in accuracy.

2. Improve the estimates of the average properties of the foundation soils for the stipulated design ground motion. This would require both laboratory tests on undisturbed samples from the site and studies of wave propagation for the site. The laboratory tests are needed to establish the actual variations with shearing strain amplitude of the shear modulus and damping capacity of the soil, whereas the wave propagation studies are needed to establish realistic values for the predominant soil strains induced by the design ground motion.

3. Incorporate the effects of interaction for the higher modes of vibration of the structure, either approximately by application of the procedures recommended in Bielak (1976), Roesset et al. (1973), and Tsai (1974) or by more precise analyses of the structure-soil system. The latter analyses may be implemented either in the time domain by application of the impulse response functions presented in Veletsos and Verbic (1974). However, the frequency domain analysis is limited to systems that respond within the elastic range while the approach involving the use of the impulse response functions is limited, at present, to soil deposits that can adequately be represented as a uniform elastic halfspace. The effects of yielding in the structure and/or supporting medium can be considered only approximately in this approach by representing the supporting medium by a series of springs and dashpots whose properties are independent of the frequency of the motion and by integrating numerically the governing equations of motion (Parmelee et al., 1969).
4. Analyze the structure-soil system by finite element method (for example, Lysmer et al., 1981; Borja et al., 1992), taking due account of the nonlinear effects in both the structure and the supporting medium.

It should be emphasized that, while these more elaborate procedures may be appropriate in special cases for design verification, they involve their own approximations and do not eliminate the uncertainties that are inherent in the modeling of the structure-foundation-soil system and in the specification of the design ground motion and of the properties of the structure and soil.

REFERENCES

Anderson, D. G., and F. E. Richart, Jr. 1976. "Effects of Straining on Shear Modulus of Clays." *Journal of the ASCE Geotechnical Engineering Division* 102 (679):975-987.

Arias, A., and R. Husid. 1965. "Influence of Damping on Earthquake Response of Structures." *Revista del IDIEM* 1 (3). In Spanish.

Ballard, R. F., and F. G. McLean. 1975. "Seismic Field Methods for In Situ Moduli." In *Proceedings of the Conference on In Situ Measurement of Soil Properties*, Vol. I, pp. 121-150. North Carolina State University.

Bernal, D. 1987. "Amplification Factors for Inelastic Dynamic *P*-delta Effects in Earthquake Analysis." *Earthquake Engineering and Structural Dynamics*, 15:835-881.

Bielak, J. 1976. "Modal Analysis for Building-Soil Interaction." *Journal of the ASCE Engineering Mechanics Division* 102 (EM5):771-786.

Bielak, J. 1975. "Dynamic Behavior of Structures with Embedded Foundations." In *Earthquake Engineering and Structural Dynamics*, Vol. 3, pp. 259-274.

Bonowitz, D., Youssef, N., and Gross, J. L., "A survey of steel moment-resisting frames buildings affected by the 1994 northridge earthquake," Report no. *NISTIR 5625*, NIST, Gaithersbur, MD, 1995.

Borja, R. I., H. A. Smith, W-H. Wu, and A. P. Amies. 1992. *A methodolgy for nonlinear soil-structure interaction effects using time-domain analysis techniques,* Report 101. Stanford, California: Blume Earthquake Engineering Center.

Chang, G.A. and Mander, J.B., “Seismic Energy Based Fatigue Damage Analysis of Bridge Columns: Part II-Evaluation of Seismic Demand,” NCEER-94-0013, National Center for Earthquake Engineering Research, State University of New York at Buffalo, 1994.

Chopra, Anil K., *Dynamics of Structures: Theory and Application to Earthquake Engineering*, Prentice Hall, Englewod Cliffs, New Jersey, 1995.

Chopra, A. K., and J. A. Gutierrez. 1974. "Earthquake Analysis of Multistory Buildings Including Foundation Interaction." *Journal of Earthquake Engineering and Structural Dynamics* 3:65-67.

Clough, R. W. and J. Penzien. 1975. *Dynamics of Structures*. New York: McGraw-Hill.

Earthquake Engineering Research Institute. 1994. *Preliminary Reconnaisance Report: Northridge Earthquake*, EERI 94-01.

Elsabee, F., I. Kausel, and J. M. Roesset. 1977. "Dynamic Stiffness of Embedded Foundations." In *Proceedings of the ASCE Second Annual Engineering Mechanics Division Specialty Conference*, pp. 40-43.

Erden, S. M. 1974. Influence of Shape and Embedment on Dynamic Foundation Response. Thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, University of Massachusetts, Amherst.

Gazetas, G. 1991. “Foundation Vibrations,” in *Foundation Engineering Handbook,* 2nd edition, edited by H.-Y. Fang .

Goel, R. K., and A. K. Chopra. 1998. “Period Formulas for Concrete Shear Wall Buildings.” *Journal of Structural Engineering*, ASCE, April.

Goel, R. K., and A. K. Chopra. 1997. “Period Formulas for Moment-Resisting Frame Buildings.” *Journal of Structural Engineering*, ASCE, November.

Hara, A., T. Ohta, M. Niwa, S. Tanaka, and T. Banno. 1974. "Shear Modulus and Shear Strength of Cohesive Soils." *Journal of the Japanese Society of Soil Mechanics and Foundation Engineering* 14 (3):1-24.

Hardin, B. O. 1978. “The Nature of Stress-Strain Behavior of Soils,” *Earthquake Engineering and Soil Dynamics*, ASCE, Vol. I, 3-90.

Hardin, B. O., and V. P. Drnevich. 1975. "Shear Modulus and Damping in Soils: Design Equations and Curves." *Journal of the ASCE Soil Mechanics and Foundation Division* 98 (SM7):667-695.

Jennings, P. C., and J. Bielak. 1973. "Dynamics of Building-Soil Interaction." *Bulletin of Seismology Society of America* 63 (1):9-48.

Kausel, E. 1974. *Forced Vibrations of Circular Foundations on Layered Media*, Report R74-11. Cambridge: Massachusetts Institute of Technology.

Kausel, E., and J. M. Roesset. 1975. "Dynamic Stiffness of Circular Foundations." *Journal of the ASCE Engineering Mechanics Division* 101 (EM6):771-785.

Kramer, S. L. 1996. *Geotechnical Earthquake Engineering*. Upper Saddle River, New Jersey: Prentice Hall.

Kuribayashi, E., T. Iwasaki, and F. Tatsuoka. 1974. "Effects of Stress Conditions on Dynamic Properties of Sands." *Bulletin of the International Institute of Seismology and Earthquake Engineering* 12:117-130.

Liu, S. C., and L. W. Fagel. 1971. "Earthquake Interaction by Fast Fourier Transform." *Journal of the ASCE Engineering Mechanics Division* 97 (EM4):1223-1237.

Luco, J. E. 1974. "Impedance Functions for a Rigid Foundation on a Layered Medium." *Nuclear Engineering and Design* (31):204-217.

Lysmer, J., M.Tabatabaie-Raissi, F. Tajirian, S. Vahdani, and F. Ostadan. 1981. *SASSI: A System for Analysis of Soil-Structures Interaction*, Report UCB/GT-81/02. Berkeley: University of California.

Miranda, E. and Bertero, V.V., "Evaluation of strength Reduction Factors for Earthquake Resistant Design," Earthquake Spectra, Vol. 10, No. 2, pp 357-380, 1994.

Nair, K., H. Gray, and N. C. Donovan. 1969. *Analysis of Pile Group Behavior.* ASTM Special Technical Publication 44.

Nassar, A.A., and Krawinkler, H., *Seismic Demands for SDOF and MDOF Systems*, Report No. 95, the John A. Blume Earthquake Engineering Center, Stanford University, Stanford, California, June, 1991.

Newmark, N. M., J. A. Blume, and K. K. Kapur. 1973. "Seismic Design Spectra for Nuclear Power Plants." *Journal of the ASCE Power Division* 99 (PO 2):873-889.

Newmark, N. M., and E. Rosenblueth. 1971. *Fundamentals of Earthquake Engineering*. New York: Prentice-Hall.

Novak, M. 1974. "Effect of Soil on Structural Response to Wind and Earthquake." In *Earthquake Engineering and Structural Dynamics*, Vol. 3, pp. 79-96.

Novak, M., and Y. O. Beredugo. 1975. "Vertical Vibrations of Embedded Footings." *Journal of the ASCE Soil Mechanics and Foundations Division* 98 (SM12):1291-1310.

Novak, M. 1974. "Dynamic Stiffness and Damping of Piles." *Canadian Geotechnical Journal* 11:574-598.

Parmelee, R. A., D. S. Perelman, and S. L. Lee. 1969. "Seismic Response of Multistory Structures on Flexible Foundations." *Bulletin of the Seismological Society of America* 29:1061-1070.

Poulos, H. G. and E. H. Davis. 1974. *Elastic Solutions for Soil and Rock Mechanics.* New York: Wiley and Sons.

Rainer, J. H. 1975. "Simplified Analysis of Dynamic Structure-Ground Interaction." *Canadian Journal of Civil Engineering* 2 (3):345-356.

Rainer, J. H. 1975. "Damping in Dynamic Structure-Foundation Interaction." *Canadian Geotechnical Journal* 12:13-25.

Richart, F. E., J. R. Hall, Jr., and R. D. Woods. *Vibrations of Soils and Foundations.* Englewood Cliffs, New Jersey: Prentice-Hall.

Roesset et al. 1973.

Rutenburg, A. 1985. "Simplified *P*-delta Analysis for Asymmetric Structures." *Journal of the Structural Division, ASCE,* 108(ST9).

SAC Joint Venture. 1995. *Analytical Investigation of Buildings Affected by the 1994 Northridge Earthquake,* Volumes 1 and 2, SAC 95-04A and B. SAC Joint Venture, Sacramento, CA, 1995.

Seed, H. B., R. T. Wong, I. M. Idriss, and K. Tokimatsu. 1984. "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils." *Rpt. No. UCB/EERC-84/14,* Earthquake Engineering Research Center, Univ. of California, Berekely.

Seed, H. B., J. Lysmer, and R. Hwang. 1974. "Soil-Structure Interaction Analysis for Seismic Response." *Journal of the ASCE Geotechnical Engineering Division* 101 (GT5):439-457.

Seed, H. B., R. V. Whitman, and J. Lysmer. 1977. "Soil-Structure Interaction Effects in the Design of Nuclear Power Plants." In *Structural and Geotechnical Mechanics, A Volume Honoring N. M. Newmark,* edited by W. J. Hall. Englewood Cliffs, New Jersey: Prentice-Hall.

Stokoe, K. H., II, and R. D. Woods. 1975. "In Situ Shear Wave Velocity by Cross-Hole Method." *Journal of the ASCE Soil Mechanics and Foundations Division* 98 (SM5):443-460.

Structural Engineers Association of California. 1968, 1973, 1974, 1976, and 1987. *Recommended Lateral Force Requirements and Commentary.* San Francisco: SEAOC.

Thomson, W. T. 1965. *Vibration Theory and Application.* New York: Prentice-Hall.

Tomlinson, M..J. 1994. *Pile Design and Consturction Practice,* E&FN Spon, London, UK, 4th Edition.

Tsai, N. C. 1974. "Modal Damping for Soil-Structure Interaction." *Journal of the ASCE Engineering Mechanics Division* 100 (EM2):323-341.

Uang, C-M., "Establishing R (or R_w) and C_d Factors for Building Seismic Provisions," *Journal of Structural Engineering,* vol 117, no. 1, pp. 19-28, ASCE, 1991.

Vaish, A. K., and A. K. Chopra. 1974. "Earthquake Finite Element Analysis of Structure-Foundation Systems." *Journal of the ASCE Engineering Mechanics Division* 100 (EM6):1011-1016.

Veletsos, A. S., and V. V. Nair. 1975. "Seismic Interaction of Structures on Hysteretic Foundations." *Journal of the ASCE Structural Division* 101 (ST1):109-129.

Veletsos, A. S. 1977. "Dynamics of Structure-Foundation Systems." In *Structural and Geotechnical Mechanics, A Volume Honoring N. M. Newmark,* edited by W. J. Hall, pp. 333-361. Englewood Cliffs, New Jersey: Prentice-Hall.

Veletsos, A. S., and J. W. Meek. 1974. "Dynamic Behavior of Building Foundation Systems." *Earthquake Engineering and Structural Dynamics* 3 (2):121-138.

Veletsos, A. S., and B. Verbic. 1973. "Vibration of Viscoelastic Foundations." *Earthquake Engineering and Structural Dynamics* 2 (1): 87-105.

Veletsos, A. S., and Y. T. Wei. 1971. "Lateral and Rocking Vibration of Footings." *Journal of the ASCE Soil Mechanics and Foundations Division* 97 (SM9):1227-1248.

Veletsos, A. S., and B. Verbic. 1974. "Basic Response Functions for Elastic Foundations." *Journal of the ASCE Engineering Mechanics Division* 100 (EM2):189-205.

Vidic, T., Fajfar, P., and Fischinger, M. (1992) "A Procedure for Determining Consistent Inelastic Design Spectra," Proc. Workshop of Nonlinear Seismic Analysis of RC Structures, Bled, Slovenia, July, 1992.

Vucetic, M., and R. Dobry. 1991. "Effect of Soil Plasticity on Cyclic Response." *J. Geotech. Engrg.* ASCE, 117(1), 89-107.

Wallace, J. W., J. P. Stewart, and A. S. Whittaker, editors. 1999. *Building Vulnerability Studies: Modeling and Evaluation of Tilt-up and Steel Reinforced Concrete Buildings*, Report to PEER-PG&E Cooperative Research Program. Los Angeles: University of California Department of Civil and Environmental Engineering.

Wiegel, R. L., Ed. 1970. *Earthquake Engineering*. New York: Prentice-Hall.

Wilson, E. L., and A. Habibullah. 1987. "Static and Dynamic Analysis of Multi-story Buildings Including *P*-delta Effects." *Earthquake Spectra* 3(2).

Wood, S. L., "Performance of Reinforced Concrete Buildings During the 1985 Chile Earthquake," *EERI Spectra*, Nov. 1991

Appendix to Chapter 5

NONLINEAR STATIC ANALYSIS PROCEDURE

C5A.1 NONLINEAR STATIC ANALYSIS PROCEDURE: The analysis procedure is intended to provide a simplified approach for directly determining the nonlinear response behavior of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism and initiation of collapse. Response behavior is gauged through measurement of the strength of the structure, at various increments of lateral displacement. The strength is measured by the shear forces resisted by a structure in the form of lateral forces, which cause the lateral deformations.

Usually the shear resisted by the system when the first element yields in the structure, although not always relevant for the entire structure, is defined as the "elastic strength." When traditional linear methods of design are used, together with R factors, the value of the design base shear sets the minimum strength at which this elastic strength point can occur.

If a structure is subjected to larger lateral loads, then represented by the elastic strength, than a number of elements will yield, eventually forming a mechanism. For most structures, multiple configurations of mechanisms are possible. The mechanism caused by the smallest set of forces is likely to appear before others do. That mechanism is considered to be the dominant mechanism. Standard methods of plastic or "limit" analysis can be used to determine the strength corresponding to such mechanisms. However, such "limit analysis" cannot determine the deformation at the onset of such a mechanism. If the yielding elements are able to strain harden than the mechanism will not allow increase of deformations without some increase of lateral forces and the mechanisms is stable. Moreover, it can be considered as a flexible version of the original frame structure. Figure C5A.7-1, which shows a plot of lateral structural strength vs. deformation of a hypothetical structure, sometimes termed a pushover curve, illustrates these concepts.

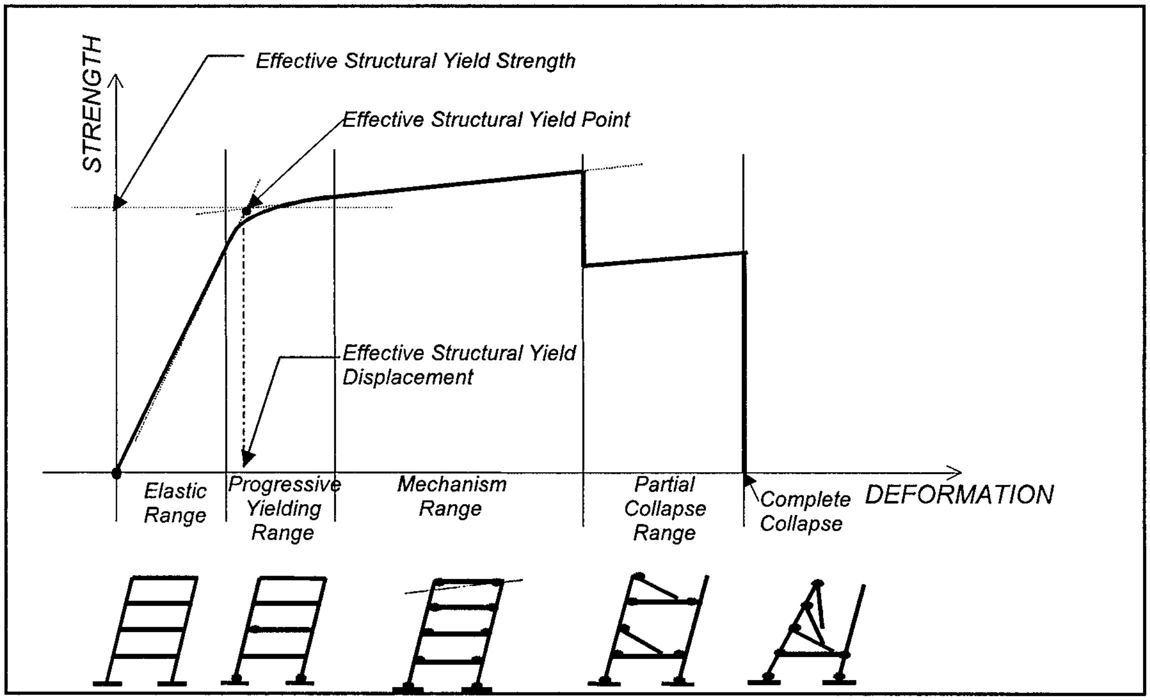


FIGURE C5A.7.1 Strength deformation relation in a frame structure.

If after the structure develops a mechanism it deforms an additional substantial amount, elements within the structure may fail, fracture, or buckle, etc., losing their strength contribution to the whole structural system. In such case, the strength of the structure will diminish with increasing deformation. If any essential element, or group of elements, fail, then the entire structure may loose capacity to carry the gravity loads, or any lateral load. This condition can also occur if the lateral deformation becomes so great that the P-delta effects exceed the residual lateral strength of the structure. Such conditions are defined as collapse and the deformation associated with collapse defined as the "ultimate deformation." This deformation can be determined by the nonlinear static procedure and also by plastic or limit analysis.

As shown in Figure C5A.7-1, many structures exhibit a range of behavior between the development of first yielding and development of a mechanism. When the structure deforms while elements are yielding sequentially (shown as progressive yielding), the relation between external forces and deformations cannot be determined by simple *limit analysis*. For such a case, other methods of analysis are required. The purpose of nonlinear static analysis is to provide a simplified method of determining structural response behavior at deformation levels intermediate to those which can be conveniently analyzed using limit state methods.

C5A.1.1 Modeling: In performing this method, the structure is modeled with elements having stiffness properties that are dependent on the amount of deformation imposed on the element. All elements than can experience deformations or forces larger than yield should be modeled with nonlinear properties. As a minimum, nonlinear stiffness properties should be described, by a bilinear model, with initial elastic stiffness, yield strength (and yield deformation), and post-yield characteristics including the point-of-loss of strength (and associated deformation) or point of complete fracture or loss of stability defined.

C5A.1.1.2 Lateral Loads: The analysis is performed by applying a monotonically increasing "set of loads" distributed throughout the structure. The analysis traces the internal distribution loads and deformations as the set of loads is progressively increased. Moreover it records the strength-deformation relation and the characteristic events occurring as the analysis progresses. The strength deformation relation typically takes a shape similar to that shown in Figure C5A.7-1.

It should be noted that nonlinear static analysis can determine the order of yielding of elements in the "progressive yielding range" (see Figure C5A.7-1) and the associated strength and deformations. The analysis can also determine the deformations associated with fractures or failure of *components* and the entire structure. However, it is accurate, only if the applied set of loads induces a pattern of deformation in the structure that is similar to that which will be induced by the earthquake ground motion. This can be controlled, to some extent, through application of an appropriate pattern of loads. However, this method is generally limited in applicability to structures that have limited participation in higher modes.

The force deformation sequence predicted by the analysis is a function of the configuration of the set of monotonically increasing loads. In order to capture the dynamic behavior of the structure, the force-deformation relation should be properly defined as the instantaneous distribution of inertial forces when the maximum response of structure occurs. Therefore, the load configuration should be redefined at each point on the pushover curve, proportional to the instantaneous configuration of inertial forces. Such a configuration is dependent on the instantaneous modal characteristics of the structure and their combination. Since the structure is nonlinear, the instantaneous modal characteristics depend on the modified properties due to inelastic deformations, changing the load distribution at each step, accordingly.

Such use of a varying, deformation-dependent load configuration would require almost as much labor and uncertainties as application of a full nonlinear response history procedure. Such effort would be inappropriate for the simplified approach that the nonlinear static procedure is intended to provide. Therefore, the load configuration and intensity are approximated in the nonlinear static procedures. Several approximations are available:

(a) An approximate distribution proportional to the idealized elastic response model as used in the equivalent lateral force method:

$$F_i = \frac{W_i h_i^k}{\sum\limits_j W_j h_i^k} V \qquad \text{(C5A.1.2-1)}$$

where, F, W, h and V are the story inertia force, the story weight and height, and the base shear, respectively; k is a power index ranging between 1 and 2 as defined in ATC3-06.

(b) A better approximation is obtained if the dominant mode of vibration is known, such as the first mode in moderate height building structures:

$$F_i = \frac{W_i \varphi_i}{\sum\limits_i w_i \varphi_i} V \qquad \text{(C5A.1.2-2)}$$

where, ϕ_i is the dominant mode shape. This approximation allows the three-dimensional distribution of inertia forces to be obtained when such considerations are important.

(c) A still more complete approximation can be obtained, if several significant modes of vibration are also known. In such cases the modes for which the total equivalent modal mass exceed 90 percent should be included. The load configuration is given by:

$$F_i = V \frac{W_i \varphi_{id}}{\Sigma w_i \varphi_{id}} \frac{\left[\Sigma\left[(\Gamma_i / \Gamma_d)(S_{ai} / S_{ad})\right]^2\right]^{1/2}}{\left[\Sigma\left[(\Gamma_i / \Gamma_d)^2 (S_{ai} / S_{ad})\right]^2\right]^{1/2}} \tag{C5A.1.2-3}$$

where, Γ_i/S_{ai} are the modal participation factor and the spectral acceleration, respectively, and subscript d indicates the dominant mode. ($\Gamma_t = \Sigma W_i \varphi_i$; where the mode shapes are ϕ are mass normalized, i.e. $\Sigma W_i \varphi_i^2 / g = 1$).

(d) If more accurate definition of the load is necessary then the configuration described by Eq. (C5A.1.2-3) should be calculated and reevaluated when changes occur in the modal characteristics of the structure as it yields. Such procedure has also defined as "adaptablepush-over."

The *Provisions* adopt the simplest of these approaches, indicated as (a) above, though the use of the more complex approaches should not be preluded. Nonlinear static analysis in several commercially available and public domain nonlinear analysis platforms.

C5A.1.3 Limit Deformation: The nonlinear analysis should be continued by increasing the loading set until the deflections at the control point exceeds 150 percent of the expected inelastic deflection. The expected inelastic deflection at each level shall be determined by combining the elastic modal values as obtained from Sec. 5.5.5 and 5.5.6 multiplied by the factor

$$C_i = \frac{(1 - T_s / T_1)}{R_d} + (T_s / T_1) \tag{C5A.1.3-1}$$

where T_s is the characteristic period of the response spectrum, defined as the period associated to the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum and R_d is the ratio of the total design base shear to the fully yielded strength of the major mechanism which can be obtained according to $R_d = R/\Omega_o$, with R and Ω_o given in Table 5.2.2. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or by the complete quadratic combination technique.

The recommendation linking the expected inelastic deformation to the elastic is based on an approach originally suggested by Newmark and on later studies by several other researchers. These are described below:

In a 1991 study, Nassar and Krawinkler published simplified expressions that were derived from the study of mean strength reduction factors computed from fifteen ground motions recorded in the

Western United States. The records used were obtained at alluvium and rock sites. The influence of the site conditions was not explicitly considered. The sensitivity of mean strength reduction factors to the epicenter distance, yield level, strain-hardening ratio ans the stiffness degradation was examined. The study concluded that epicentral distance and stiffness degradation have negligible influence on strength reduction factors. Ratios of inelastic displacements to displacements predicted by elastic analysis were derived from the above work:

$$R_d = \left[1 + \frac{1}{c}\left(r^c - 1\right)\right] / r \cdot \geq 1 \qquad \text{(C5A.1.3-1)}$$

$$c = \frac{T^a}{1 + T^a} + \frac{b}{T} \qquad \text{(C5A.1.3-2)}$$

In the above, T, is the period of vibration of the structure and r is the strength ratio. R_d defined above and used in the NEHRP guidelines.

In 1994, Chang and Mander performed analytical studies based on an envelope of five recorded ground motions. An inelastic dynamic magnification factor that relates the maximum inelastic displacement to the elastic spectral displacement was obtained.

$$R_D = \left(1 - \frac{1}{r}\right)\left(\frac{T_{PV}}{T}\right)^n + \frac{1}{r} \cdot \geq 1 \qquad \text{(C5A.1.3-3)}$$

where T_{PV} period at which the maximum spectral velocity response occurs, and

$$n = 1.2 + 0.025r \text{ for } T_{PV} \leq 1.2\,\text{sec} \qquad \text{(C5A.1.3-4.a)}$$

$$n = 1.2 \text{ for } T_{PV} > 1.2\,\text{sec} \qquad \text{(C5A.1.3-4.b)}$$

In 1992, Vidic, Fajfar, and Fischinger recommended simplified expressions derived from the study of the mean strength reduction factors computed from twenty ground motions recorded in the Western United States as well as in the 1979 Montenegro, Yugoslavia, earthquake. Systems with bilinear and stiffness degrading (Q-model) hysteric behavior and viscous damping proportional to the mass and the instantaneous stiffness were considered.

$$R_D = \left(1 - \frac{1}{r}\right)\frac{T_0}{T} + \frac{1}{r} \cdot \geq 1 \qquad \text{(C5A.1.3-5)}$$

where T is the dominant period of structure and $T_0 = 0.65\mu^{0.3} T_1$

$$T_1 = 2\pi \frac{\varphi_{ev}}{\varphi_{ea}} \frac{V}{A} \qquad \text{(C5A.1.3-6)}$$

where V and A are the peak ground velocity and peak ground acceleration, respectively. For the 20 ground motions considered in the study, the mean amplification factors ϕ_{ea} and ϕ_{ev} are 2.5 and 2.0, respectively.

Miranda and Bertero (1994) suggested simplified expressions derived from the study of the mean strength reduction factors computed from 124 ground motions recorded on a wide range of soil conditions. The study considered 5 percent damped bilinear systems undergoing displacement ductility ratios between 2 and 6. Based on the local site conditions at the recording station, ground motions were classified into three groups; rock sites, and soft soil sites. In addition to the influence of soil conditions, the study considered the influence of magnitude and epicentral distance on strength reduction factors. The study concluded that soil conditions influence the reduction factors significantly (particularly for soft soil sites); on the other hand, magnitude and epicenter distance have a negligible effect on mean strength reduction factors.

$$R_D = \left(1 - \frac{1}{r}\right)\Phi + \frac{1}{r}. \qquad \text{(C5A.1.3-7)}$$

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp\left[-\frac{3}{2}\left(\ln T - \frac{3}{5}\right)^2\right] \qquad \text{(C5A.1.3-8)}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2\left(In T - \frac{1}{5}\right)^2\right] \qquad \text{(C5A.1.3-9)}$$

$$\Phi = 1 + \frac{T_g}{3T} - \frac{3T_g}{2T}\exp\left[-3\left(\ln\frac{T}{T_g} - \frac{1}{4}\right)^2\right] \qquad \text{(C5A.1.3-10)}$$

where T is the period of vibration of the structure and T_g is the characteristic ground motion period.

The recommended formulation contained in the *Provisions* is a combination of the recommendations of Krawinkler et al and of Vidic et al with some simplification. The inaccuracy is covered by the request of 50 percent accedence of the calculated target. In addition the 50 percent margin is required since a small variation in strength (due to modeling or due to imprecise construction) can lead to large displacement variations in the inelastic range.

Chapter 6 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS

6.1 GENERAL: The general requirements establish minimum design levels for architectural, mechanical, electrical, and other nonstructural systems and *components* (hereinafter referred to as "*components*") recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural *components*. Several exemptions are made to the *Provisions*:

1. All *components* in Seismic Design Category A are exempted because of the lower seismic input for these items

2. All mechanical and electrical *components* in Seismic Design Categories B and C are exempted if they have an importance factor (I_p) equal to 1.00 because of the low acceleration and the classification that they do not contain hazardous substances and are not required to function to maintain life-safety.

3. All *components* in all Seismic Design Categories, weighing less than 400 pounds (1780 N), and are mounted 4 ft (1.22 m) or less above the floor are exempted if they have an importance factor (I_p) equal to 1.00, because they do not contain hazardous substances, are not required to function to maintain life safety, and are not considered to be mounted high enough to be a life-safety hazard if they fell.

The seismic force on any *component* shall be applied at the center of gravity of the *component* and shall be assumed to act in any horizontal direction. Vertical forces on architectural *components* are specified in Sec. 6.1.3. Vertical forces on mechanical and electrical *components* are specified in Sec. 6.3.2.

In the design and evaluation of support structures and the attachment of the architectural *component*, flexibility should be considered. *Components* that are subjected to seismic relative displacements (i.e., *components* that are connected to both the floor and ceiling level above) should be designed with adequate flexibility to accommodate imposed displacements. In the design and evaluation of equipment support structures and attachments, flexibility will reduce the fundamental frequency of the supported equipment and increase the amplitude of its induced relative motion. This lowering of the fundamental frequency of the supported *component* often will bring it into the range of the fundamental frequency of the supporting building or into the high energy range of the input motion. In evaluating the flexibility/stiffness of the *component* attachment, the load path in the *components* should be considered especially in the region near the anchor points.

Although the *components* included in Tables 6.2.2 and 6.3.2 are listed separately, significant interrelationships exist among them and should not be overlooked. For example, exterior, nonstructural, spandrel walls may shatter and fall on the streets or walks below seriously hampering accessibility and egress functions. Further, the rupture of one *component* could lead to the failure of another that is dependent on the first. Accordingly, the collapse of a single *component* ultimately may lead to the failure of an entire system. Widespread collapse of suspended ceilings and light fixtures in a building may render an important space or major exit stairway unusable.

Consideration also was given to the design requirements for these *components* to determine how well they are conceived for their intended functions. Potential beneficial and/or detrimental interactions with the structure were examined. The interrelationship between *components* and their attachments were surveyed. Attention was given to the performance relative to each other of architectural, mechanical, and electrical *components*; building products and finish materials; and systems within and without the building structure. It should be noted that the modification of one *component* in Table 6.2.2 or 6.3.2 could affect another and, in some cases, such a modification could help reduce the risk associated with the interrelated unit. For example, landscaping barriers around the exterior of certain buildings could decrease the risk due to falling debris although this should not be interpreted to mean that all buildings must have such barriers.

The design of *components* that are in contact with or in close proximity to structural or other nonstructural *components* must be given special study to avoid damage or failure when seismic motion occurs. An example is where an important element, such as a motor generator unit for a hospital, is adjacent to a nonload-bearing partition. The failure of the partition might jeopardize the motor generator unit and, therefore, the wall should be designed for a performance level sufficient to ensure its stability.

Where nonstructural wall *components* may affect or stiffen the structural system because of their close proximity, care must be exercised in selecting the wall materials and in designing the intersection details to ensure the desired performance of each *component*.

6.1.2 Component Force Transfer: It is required that *components* be attached to the *structure* and that all the required attachments be fully detailed in the design documents, or be specified in accordance with approved standards. These details should take into account the force levels and anticipated deformations expected or designed into the structure.

The calculation of forces as prescribed in Sec. 6.1.3 recognizes the unique dynamic and structural characteristics of the *components* as compared to *structures*. *Components* typically lack attributes of *structures*, i.e., ductility, toughness, and redundancy, which factor in to the calculation of reduced lateral design forces. This is reflected in the lower values for R_p given in Tables 6.2.2 and 6.3.2, as compared to R values for *structures*. In addition, *components* may exhibit unique dynamic amplification characteristics, as reflected in the values for a_p in Tables 6.2.2 and 6.3.2. Thus, for the calculation of the *component* integrity and connection to the supporting *structure*, greater forces are used, as a percentage of *component* mass, than are typically calculated for the overall lateral load resisting system. It is the intent of this provision that *component* forces be accommodated in the *structure* design as required to prevent local overstress of the immediate vertical- and lateral-load carrying systems. Inasmuch as the *component* masses are included, explicitly or otherwise, in the design of the lateral load resisting system, it is generally sufficient for verification of a complete load path to only check for local overstress conditions in the vicinity of the *component* in question. Where *component* forces have increased due to the nature of the anchorage system, these load increases, which take the form of reductions in R_p, or increase of F_p, need not be considered in the check of the load path.

An area of concern that is often overlooked is the reinforcement and positive connection of housekeeping slabs to the supporting *structure.* Lack of such reinforcement and connections has led to costly failures in past earthquakes. Therefore, the housekeeping slabs must be considered as part of the continuous load path, and be positively fastened to the supporting *structure.*

For the purposes of the load path check, it is essential that detailed information on the *components*, including size, weight, and location of *component* anchors, be communicated to the *registered design professional* responsible for the *structure* during the design process. Note, until the *component* is ordered, the exact size and location of loads will generally not be known. Therefore, the designer should make conservative assumptions in the design of the supporting structural elements. The design of the elements must be checked, once the final magnitude and location of the design loads have been established.

If an architectural *component* were to fail during an earthquake, the mode of failure probably would be related to faulty design of the *component,* interrelationship with another *component* that fails, interaction with the structural framing, deficiencies in its type of mounting, or inadequacy of its attachments or anchorage. The last is perhaps the most critical when considering seismic safety.

Building *components* designed without any intended structural function--such as infill walls--may interact with the structural framing and be forced to act structurally as a result of excessive building deformation. The build up of stress at the connecting surfaces or joints may exceed the limits of the materials. Spatial tolerances between such *components* thus become a governing factor. These requirements therefore emphasize the ductility and strength of the attachments for exterior wall elements and the interrelationship of elements.

Traditionally, mechanical equipment that does not include rotating or reciprocating *components* (e.g., tanks, heat exchangers) is anchored directly to the building structure. Mechanical and electrical equipment containing rotating or reciprocating *components* often is isolated from the structure by vibration isolators (rubber-in-shear, springs, air cushions). Heavy mechanical equipment (e.g., large boilers) often is not restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, usually is rigidly anchored (e.g., switchgear, motor control centers). The installation of unattached mechanical and electrical equipment should be virtually eliminated for buildings covered by the *Provisions.*

Friction produced solely by the effects of gravity cannot be counted on to resist seismic forces as equipment and fixtures often tend to "walk" due to rocking when subjected to earthquake motions. This often is accentuated by the vertical ground motions. Because frictional resistance cannot be relied upon, positive restraint must be provided for each *component.*

6.1.3 Seismic Forces: The design seismic force is dependent upon the weight of the system or *component*, the *component* amplification factor, the *component* acceleration at point of attachment to the structure, the *component* importance factor, and the *component* response modification factor.

The seismic design force equations presented originated with a study and workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) with funding from the National Science Foundation (NSF) (Bachman et al., 1993). The participants examined recorded acceleration data in response to strong earthquake motions. The objective was to develop a "supportable" design force equation that considered actual earthquake data as well as *component* location in the structure, *component* anchorage ductility, *component* importance, *component* safety hazard upon separation from the structure, structural response, site conditions, and seismic zone. Additional studies have further revised the equation to its present form (Drake and Bachman, 1994 and 1995). In addition, the term C_a has been replaced by the quantity $0.4S_{DS}$ to conform with changes in Chapter 4. BSSC Technical Subcommittee 8 believes that Eq. 6.1.3-1 through 6.1.3-3 achieve the objectives without unduly burdening the practitioner with complicated formulations.

The *component* amplification factor (a_p) represents the dynamic amplification of the *component* relative to the fundamental period of the structure (T). It is recognized that at the time the *components* are designed or selected, the structural fundamental period is not always defined or readily available. It is also recognized that the *component* fundamental period (T_p) is usually only accurately obtained by expensive shake-table or pull-back tests. A listing is provided of a_p values based on the expectation that the *component* will usually behave in either a rigid or flexible manner. In general, if the fundamental period of the *component* is less than 0.06 sec, no dynamic amplification is expected. It is not the intention of the *Provisions* to preclude more accurate determination of the *component* amplification factor when reasonably accurate values of both the structural and *component* fundamental periods are available. Figure C 6.1.3-1 is from the NCEER work and is an acceptable formulation for a_p as a function of T_p/T. Minor adjustments from the 1994 *Provisions* have been made in the tabulated a_p values to be consistent with the 1997 *Uniform Building Code.*

The *component* response modification factor (R_p) represents the energy absorption capability of the *component*'s structure and attachments. Conceptually, the R_p value considers both the overstrength and deformability of the *component*'s structure and attachments. In the absence of current research, it is believed these separate considerations can be adequately combined into a single factor. The engineering community is encouraged to address the issue and conduct research into the *component* response modification factor that will advance the state of the art. These values are judgmentally determined utilizing the collective wisdom and experience of the responsible committee. In general, the following benchmark values were used:

R_p =1.5, low deformability element
R_p = 2.5, limited deformability element
R_p = 3.5, high deformability element

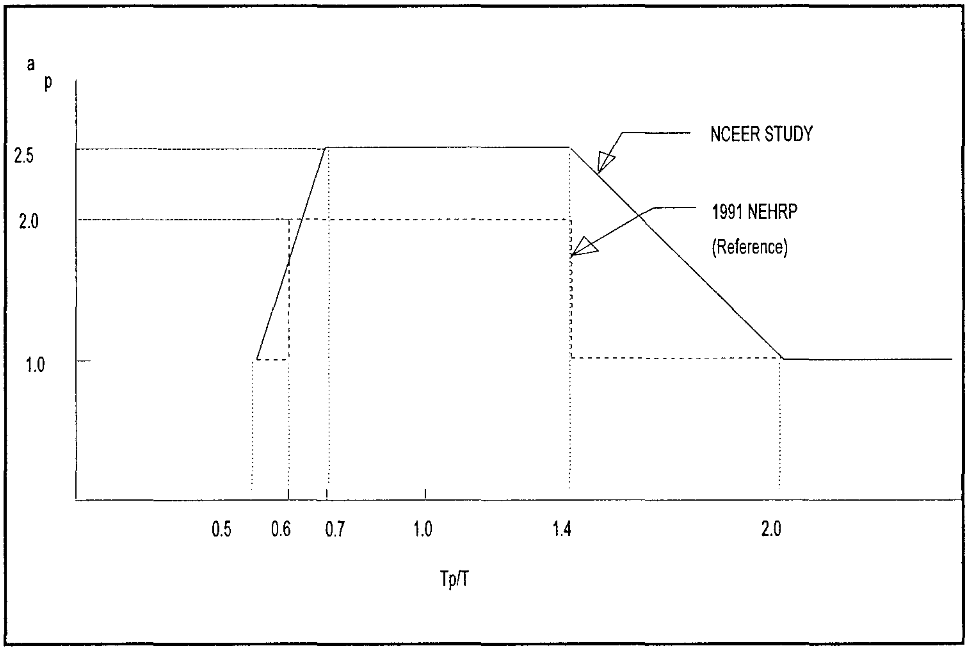


FIGURE C6.1.3-1 NCEER formulation for a_p as function of structural and component periods.

Minor adjustments from the 1994 *Provisions* have been made in the tabulated R_p values to correlate with F_p values determined in accordance with the 1997 *Uniform Building Code.* Researchers have proposed a procedure for validating values for R_p with respect to documented earthquake performance (Bachman and Drake, 1996).

Eq. 6.1.3-1 represents a trapezoidal distribution of floor accelerations within the structure, linearly varying from the acceleration at the ground ($0.4S_{DS}$) to the acceleration at the roof ($1.2S_{DS}$). The ground acceleration ($0.4S_{DS}$) is intended to be the same acceleration used as design input for the structure itself and will include site effects.

Examination of recorded in-structure acceleration data in response to large California earthquakes reveals that a reasonable maximum value for the roof acceleration is four times the input ground acceleration to the structure. Earlier work (Drake and Bachman, 1996, 1995 and 1996) indicated that the maximum amplification factor of four seems suitable (Figure C6.1.3-2). However, a close examination of recently recorded strong motion data at sites with peak ground accelerations in excess of 0.1g indicates that an amplification factor of three is more appropriate (Figure C6.1.3-3). In the lower portions of the structure (the lowest 20 percent of the structure), both the amplification factors of three and four do not bound the mean plus one standard deviation accelerations. However, the minimum design force in Eq. 6.1.3-3 provides a lower bound in this region.

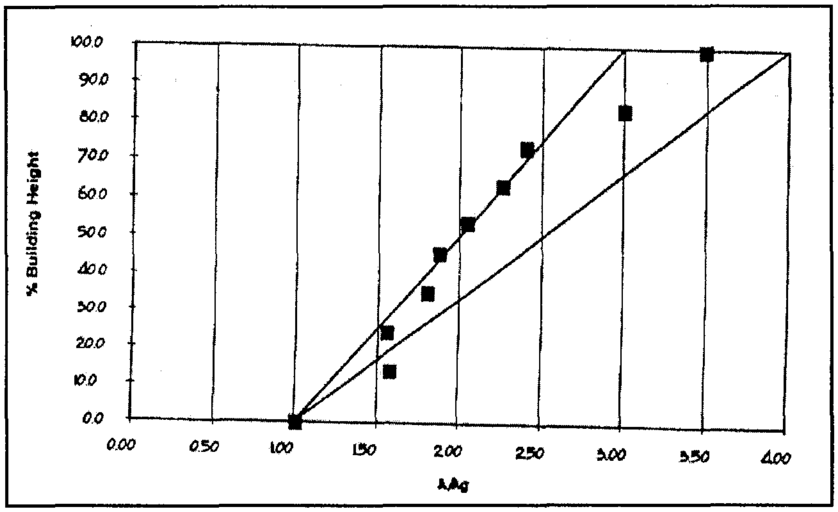


FIGURE C6.1.3-1 Revised NEHRP equation vs (Mean + 1σ) acceleration records -all sites.

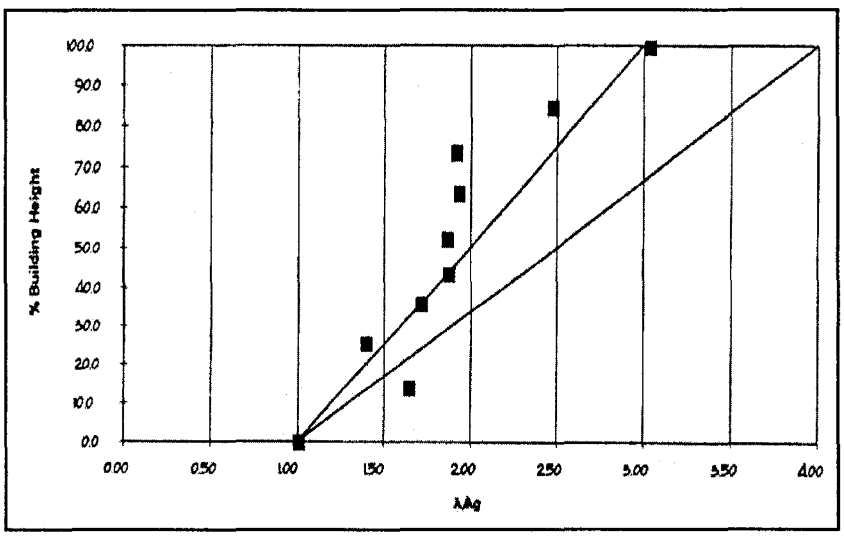


FIGURE C6.1.3-3 Revised NEHRP equation vs (Mean +1σ) acceleration records - sites with A_g ≥0.1g.

Examination of the same data indicates that the in-structure accelerations do not decrease with larger building periods as might be expected from reviewing typical response spectra. One reason for invalidating the traditional response spectra shape might be that structures with longer fundamental periods may have designs governed by drift requirements. These structures would be stiffer with more elastic capacity and also may have lower damping at higher acceleration responses. Also, site soil amplifications are greater at longer periods than at shorter periods. As a result of these studies, the structural period effect introduced into the 1994 *Provisions* for *components* has been removed from the 1997 *Provisions*.

A lower limit for F_p is set to assure a minimal seismic design force. The minimum value for F_p determined by setting the quantity a_pA_p/R_p equal to0.7C_a which is equivalent to the minimum used in current practice. In addition, the C_a term was converted to 0.4S_{DS} to be consistent with changes to Chapter 1. The resultant multiplication of 0.7 times 0.4 equals 0.28 was rounded to 0.3 for simplicity.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 6.1.3-2is the maximum value for F_p determined by setting the quantity a_pA_p/R_p equal to 4.0. In addition, the term C_a was converted to 0.4 S_{DS} to be consistent with changes to Chapter 4. Eq. 6.1.3-2also serves as a reasonable "cutoff" equation to assure that the multiplication of the individual factors does not yield an unreasonably high design force.

To clarify the application of vertical seismic design forces in combination with horizontal design forces and service loads, a cross-reference was provided to Sec. 2.2.6. The value for F_p calculated in accordance with Chapter 6 should be substituted for the value of Q_E in Sec. 2.2.6.

For elements with points of attachment at varying heights, it is recommended that $\mathrm{F_p}$ be determined individually at each height (including minimums) and the values averaged.

Alternatively for each point of attachment a force F_p shall be determined based on Eq. 6.1.3-1. Minimums and maximums of Eq. 6.1.3 shall be utilized in determining each F_p . The weight W_p used in determining each F_p should be based on the tributary weight of the *component* associated with the point of attachment. For designing the *component*, the attachment force F_p should be distributed relative to the *components* mass distribution over the area used to establish the tributary weight (e.g. for tilt-up walls, a uniform horizontal load would be applied half-way up the wall equal to F_p min.) With the exception of out-of-plane wall anchorage to flexible diaphragms which is covered by Eq. 5.2.6.3.3, each anchorage force should be based on simple statics determined using all the distributed loads applied to the complete *component*. Cantilever parapets that are part of a continuous element should be separately checked for parapet forces.

6.1.4 Seismic Relative Displacements: The seismic relative displacement equations were developed as part of the NCEER/NSF study and workshop described above. It was recognized that displacement equations were needed to support the design of cladding, stairwells, windows, piping systems, sprinkler *components*, and other *components* that are connected to the structure(s) at multiple levels or points of connection.

Two equations are given for each situation. Eq. 6.1.4-1 and Eq. 6.1.4-3 yield "real" structural displacements as determined by elastic analysis, with no structural response modification factor (R) included. Recognizing that elastic displacements are not always defined or available at the time the *component* is designed or procured, default Eq. 6.1.4-2 and Eq. 6.1.4-4 also are provided that allow the use of structure drift limitations. Use of these default equations must balance the need for a timely *component* design/procurement with the possible conservatism of their use. It is the intention that the lesser of the paired equations be acceptable for use.

The designer also should consider other situations where seismic relative displacements could impose unacceptable stresses on a *component* or system. One such example would be a *component* connecting two pieces of equipment mounted in the same building at the same elevation, where each piece of equipment has its own displacements relative to the mounting location. In this case, the designer must accommodate the total of the separate seismic displacements relative to the equipment mounting location.

For some items such as ductile piping, relative seismic displacements between support points generally are of more significance than forces. Piping made of ductile materials such as steel or copper can accommodate relative displacements by local yielding but with strain accumulations well below failure levels. However, *components* made of less ductile materials can only accommodate relative displacement effects by use of flexible connections or avoiding local yielding. It is further the intent of the *Provisions* to consider the effects of seismic support relative displacements and displacements caused by seismic force on mechanical and electrical *component* assemblies such as piping systems, cable and conduit systems, and other linear systems, most typically, and the equipment to which they attach. Impact of *components* should also be avoided although ductile materials have been shown to be capable of accommodating fairly significant impact loads. With protective coverings, ductile mechanical and electrical *components* and many more fragile *components* can be expected to survive all but the most severe impact loads.

6.1.5 Component Importance Factor: The *component* importance factor (I_p) represents the greater of the life-safety importance of the *component* and the hazard exposure importance of the structure. This factor indirectly accounts for the functionality of the *component* or structure by requiring design for a higher force level. Use of higher I_p requirements together with application of the requirements in Sec. 6.3.13 and 6.3.14 should provide better, more functional *component.* While this approach will provide a higher degree of confidence in the probable seismic performance of a *component*, it may not be sufficient for all *components*. For example, individual ceiling tiles may still fall from the ceiling grid. Seismic qualification approaches presently in use by the Department of Energy (DOE) and the Nuclear Regulatory Commission (NRC) should be considered by the registered design professional and/or the owner when unacceptable consequences of failure are anticipated.

Components that could fall from the structure are among the most hazardous building *components* in an earthquake. These *components* may not be integral with the structural system and may cantilever horizontally or vertically from their supports. Critical issues affecting these *components* include their weight, their attachment to the structure, their breakage characteristics (glass) and their location (over an entry or exit, public walkway, atrium, or lower adjacent structure). Examples of items that may pose a falling hazard include parapets, cornices, canopies, marquees, glass, and precast concrete cladding panels. In addition, mechanical and electrical *components* may pose a falling hazard, for example, a rooftop tank or cooling tower, which if separated from the structure, will fall to the ground.

Special consideration should be given *components* that could block means of egress or exitways apply to items that, if they fall during an earthquake, could block the means of egress for the occupants of the structure. The term "means of egress" has been defined the same way throughout the country, since egress requirements have been included in building codes because of fire hazard. The requirements for exitways include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passage ways, exit courts, and yards. Example items that should be included when considering egress include walls around stairs, corridors, veneers, cornices, canopies, and other ornaments above building exits. In addition, heavy partition systems vulnerable to failure by collapse, ceilings, soffits, light fixtures, or other objects that could fall or obstruct a required exit. door or *component* (rescue window or fire escape) could be considered major obstructions. Examples of the *components* that do not pose a significant falling hazard include fabric awnings and canopies and architectural, mechanical, and electrical *components* which, if separated from the structure, will fall in areas that are not accessible (in an atrium or light well not accessible to the public for instance).

Sec. 1.3.1 requires that Group III structures shall, in so far as practical, be provided with the capacity to function after an earthquake. To facilitate this, all nonstructural *components* and equipment in structures in Seismic Use Group III, and in Seismic Design Category C or higher, should be designed with an I_p equal to 1.5. All *components* and equipment are included because damage to vulnerable unbraced systems or equipment may disrupt operations following an earthquake, even if they are not "life-safety" items. Nonessential items can be considered "black boxes." There is no need for *component* analysis as discussed in Sec. 6.3.13 and 6.3.14, since operation of these secondary items is not critical to the post-earthquake operability of the structure.

Until recently, storage racks were primarily installed in low-occupancy ware houses. With the recent proliferation of warehouse-type retail stores, it has been judged necessary to address the relatively greater seismic risk that storage racks may pose to the general public, compared to more conventional retail environments. Under normal operating conditions, retail stores have a far higher occupancy load than an ordinary warehouse of a reasonable size. Failure of a storage rack system in the retail environment is much more likely to cause personal injury than a similar failure in a storage warehouse. Therefore, to provide an appropriate level of additional safety in areas open to the public, Sec 6.1.5 now requires that storage racks in occupancies open to the general public should be designed with an I_p value equal to 1.50. Storage rack contents, while beyond the scope of the *Provisions* pose a potentially serious threat to life should they fall from the shelves in an earthquake. Restraints should be provided to prevent the contents of rack shelving open to the general public from falling in strong ground shaking.

6.1.6 Component Anchorage: In general, it is not recommended that anchors be relied upon for energy dissipation. Inasmuch as the anchor represents the transfer of load from a relatively deformable material (e.g., steel) to a low deformability material (e.g., concrete, masonry), the boundary conditions for ensuring deformable, energy-absorbing behavior in the anchor itself are at best difficult to achieve. On the other hand, the concept of providing a fuse, or deformable link, in the load path to the anchor is encouraged. This approach allows the designer to provide the necessary level of ductility and overstrength in the connection while at the same time protecting the anchor from overload and eliminates the need for balancing of steel strength and deformability in the anchor with variable edge distances and anchor spacings. The restriction on R_p values for shallow anchors is because of the concern for low deformation failure modes in the *component* anchorage. Anchorages that can be reasonable expected to fail in a low deformation manner should be designed using $R_p = 1.5$. Shallow anchors are defined as those anchors that have an embedment length diameter ratio of less than 8.

For purposes of the *Provisions*, chemical anchors are intended to include post installed metal fasteners which are inserted into holes in concrete or masonry and held in place by epoxy, resins or other chemicals. Adhesive anchorages are intended to include plates, angles, or other structural elements adhered to surfaces such as computer access floor base plates.

Allowable loads for anchors should not be increased for earthquake loading. Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined through appropriate dynamic testing.

Anchors that are used to support towers, masts, and equipment often are provided with double nuts to allow for leveling during installation. Where baseplate grout is provided, it should not be relied upon to carry loads since it can shrink and crack or is often omitted altogether. In this case, the anchors are loaded in tension, compression, shear, and flexure and should be designed as such. Prying forces on anchors, which result from a lack of rotational stiffness in the connected part, can be critical for anchor design and must be considered explicitly.

For anchorages that are not provided with a mechanism to transfer compression loads, the design for overturning must reflect the actual stiffness of the baseplate, equipment, housing, etc., in determining the location of the compression centroid and the distribution of uplift loads to the anchors.

Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined through appropriate dynamic testing.

While the requirements do not prohibit the use or single anchor connections, it is considered necessary to use at least two anchors in any load-carrying device whose failure might lead to collapse.

Tests have shown that there are consistent shear ductility variations between bolts anchored to drilled or punched plates with nuts and connections using welded, headed studs. Recommendations for design are not presently available but should be considered in critical connections subject to dynamic or seismic loading.

It is important to relate the anchorage demands defined by Chapter 6 with the material capacities defined in the other chapters.

6.1.6.5: Generally, powder driven fasteners in concrete tend to exhibit variations in load capacity that are somewhat larger than post-drilled anchors and do not provide the same levels of reliability even though some installation methods allow for the same reliability as post-drilled expansion anchors. As such, their qualification under a simulated seismic test program should be demonstrated prior to use. Such fasteners, when properly installed in steel, are reliable, showing high capacities with very low variability.

6.1.7 Construction Documents: It is deemed important by the committee that there be a clearly defined basis for each quality assurance activity specified in Chapter 3. As result *construction documents* are required for all *components* requiring special inspection or testing in Chapter 3.

It is also deemed important by the committee that there be some reasonable level of assurance that the construction and installation of *components* be consistent with the basis of the supporting seismic design. Of particular concern are systems involving multiple trades and suppliers. In these cases, it is important that a registered design professional prepare construction documents for the use by the multiple trades and suppliers to follow in the course of construction.

6.2 ARCHITECTURAL COMPONENT DESIGN:

6.2.1 General: The primary focus of the *Provisions* is on the design of attachments, connections, and supports for architectural *components*.

"Attachments" are means by which *components* are secured or restrained to the seismic force resisting system of the structure. Such attachments and restraints may include anchor bolting, welded connections, and fasteners.

"Architectural *component* supports" are those members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between the *component* and the building structure. Architectural *component* supports also transmit lateral forces and/or provide structural stability for the *component* to which they connect.

The requirements are intended to reduce the threat of life safety hazards posed by *components* and elements from the standpoint of stability and integrity. There are several circumstances where such *components* may pose a threat.

1. Where loss of integrity and/or connection failure under seismic motion poses a direct hazard in that the *components* may fall on building occupants.
2. Where loss of integrity and/or connection failure may result in a hazard for people outside of a building in which *components* such as exterior cladding and glazing may fall on them.
3. Where failure or upset of interior *components* may impede access to a required exit.

The requirements are intended to apply to all of the circumstances listed above. Although the safety hazard posed by exterior cladding is obvious, judgment may be needed in assessing the extent to which the requirements should be applied to other hazards.

Property loss through damage to architectural *components* is not specifically addressed in the *Provisions*. Function and operation of a building also may be affected by damage to architectural *components* if it is necessary to cease operations while repairs are undertaken. In general, requirements to improve life-safety also will reduce property loss and loss of building function.

In general, functional loss is more likely to be affected by loss of mechanical or electrical *components*. Architectural damage, unless very severe, usually can be accommodated on a temporary basis. Very severe architectural damage results from excessive structural response that often also results in significant structural damage and building evacuation.

6.2.2 Architectural Component Forces and Displacements: *Components* that could be damaged or could damage other *components* and are fastened to multiple locations of a structure should be designed to accommodate seismic relative displacements. Examples of *components* that should be designed to accommodate seismic relative displacements include glazing, partitions, stairs, and veneer.

Certain types of veneer elements, such as aluminum or vinyl siding and trim, possess high deformability. These systems are generally light and can undergo large deformations without separating from the structure. However, care must be taken when designing these elements to ensure that the low deformability *components* that may be part of the curtain wall system, such as glazing panels, have been detailed to accommodate the expected deformations without failure.

6.2.3 Architectural Component Deformation: Specific requirements for cladding are provided. Glazing, both exterior and interior, and partitions must be capable of accommodating story drift without causing a life-safety hazard. Design judgment must be used with respect to the assessment of life-safety hazard and the likelihood of life-threatening damage. Special detailing to accommodate drift for typical replaceable gypsum board or demountable partitions is not likely to be cost-effective, and damage to these *components* has a low life-safety hazard. Nonstructural fire-resistant enclosures and fire-rated partitions may require some special detailing to ensure that they retain their integrity. Special detailing should provide isolation from the adjacent or enclosing structure for deformation equivalent to the calculated drift (relative displacement). In-plane differential movement between structure and wall is permitted. Provision also must be made for out-of-plane restraint. These requirements are particularly important in relation to the larger drifts experienced in steel or concrete moment frame structures. The problem is less likely to be encountered in stiff shear wall structures.

Differential vertical movement between horizontal cantilevers in adjacent stories (i.e., cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in design of exterior walls.

6.2.4 Exterior Nonstructural Wall Elements and Connections: The *Provisions* requires that nonbearing wall panels that are attached to or enclose the structure shall be designed to resist the (inertial) forces and shall accommodate movements of the structure resulting from lateral forces or temperature change. The force requirements often overshadow the importance of allowing thermal movement and may therefore require special detailing in order to prevent moisture penetration and allow thermal movements.

Connections should be designed such that, if they were to yield, they would do so in a high deformation manner without loss of load-carrying capacity. Between points of connection, panels should be separated from the building structure to avoid contact under seismic action.

The *Provisions* document requires allowance for story drift. This required allowance can be 2 in. (51 mm) or more from one floor to the next and may present a greater challenge to the registered design professional than requirements for the forces. In practice, separations between panels are usually limited to about 3/4 in. (19 mm), with the intent of limiting contact, and hence panel alignment disruption and/or damage under all but extreme building response, and providing for practical joint detailing with acceptable appearance. The *Provisions* calls for a minimum separation of ½ in. (13 mm). The design should respect the manufacturing and construction tolerances of the materials used to achieve this dimension.

If wind loads govern, connectors and panels should allow for not less than two times the story drift caused by wind loads determined using a return period appropriate to the site location.

The *Provisions* requirements are in anticipation of frame yielding to absorb energy. The isolation can be achieved by using slots, but the use of long rods that flex is preferable because this approach is not dependent on installation precision to achieve the desired action. The rods must be designed to carry tension and compression in addition to induced flexural stresses. For floor-to-floor wall panels, the panel usually is rigidly fixed to and moves with the floor structure nearest the panel bottom. In this condition, the upper attachments become isolation connections to prevent building movement forces from being transmitted to the panels. and thus the panel translates with the load supporting structure. The panel also can be supported at the top with the isolation connection at the bottom.

When determining the length of slot or displacement demand for the connection, the cumulative effect of tolerances in the supporting frame and cladding panel must be considered.

The *Provisions* requires that fasteners be designed for approximately 4 times the required panel force and that the connecting member be ductile. This is intended to ensure that the energy absorption takes place in the connecting member and not at the connection itself and that the more brittle fasteners remain essentially elastic under seismic loading. The factor of 4 has been incorporated into the a_p and R_p factors in consideration of installation and material variability.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, the connection system generally is statically determinant. As a result, cladding panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences.

6.2.5 Out-of-Plane Bending: Most walls are subject to out-of-plane forces when a building is subjected to an earthquake. These forces and the bending they induce must be considered in the design of wall panels, nonstructural walls, and partitions. This is particularly important for systems composed of brittle materials and/or low flexural strength materials. The conventional limits based upon deflections as a proportion of the span may be used with the applied force as derived in Sec. 6.2.2.

Judgment must be used in assessing the deflection capability of the *component*. The intent is that a heavy material (such as concrete block) or an applied finish (such as brittle heavy stone or tile) should not fail in a hazardous manner as a result of out-of-plane forces. Deflection in itself is not a hazard. A steel-stud partition might suffer considerable deflection without creating a hazard; but if the same partition supports a marble facing, a hazard might exist and special detailing may be necessary.

6.2.6 Suspended Ceilings: Suspended ceiling systems usually are fabricated using a wide range of building materials with individual *components* having different material characteristics. Some systems are homogeneous whereas others incorporate suspension systems with acoustic tile or lay-in panels. Seismic performance during recent large California earthquakes has raised two concerns:

a. The support of the individual panels at walls and expansion joints, and

b. The interaction with fire sprinkler systems.

The alternate methods provided have been developed in a cooperative effort by registered design professionals, the ceiling industry, and the fire sprinkler industry in an attempt to address these concerns. It is hoped that further research and investigation will result in further improvements in future editions of the *Provisions*.

Consideration shall be given to the placement of seismic bracing and the relation of light fixtures and other loads placed into the ceiling diaphragm and the independent bracing of partitions in order to effectively maintain the performance characteristics of the ceiling system. The ceiling system may require bracing and allowance for the interaction of *components*.

Dynamic testing of suspended ceiling systems constructed according to the requirements of current industry seismic standards (*UBC* Standard 25-2) performed by ANCO Engineers, Inc. (1983) has demonstrated that the splayed wire even with the vertical compression strut may not adequately limit lateral motion of the ceiling system due to the flexibility introduced by the straightening of the wire end loops. In addition, splay wires usually are installed slack to prevent unleveling of the ceiling grid and to avoid above-ceiling utilities. Not infrequently, bracing wires are omitted because of obstructions. Testing also has shown that system performance without splayed wires or struts was good if adequate width of closure angles and penetration clearance was provided.

The lateral seismic restraint for a non-rigidly braced suspended ceiling is primarily provided by the ceiling coming in contact with the perimeter wall. The wall provides a large contact surface to restrain the ceiling. The key to good seismic performance is that the width of the closure angle around the perimeter is adequate to accommodate ceiling motion and that penetrations, such as columns and piping, have adequate clearance to avoid concentrating restraining loads on the ceiling system. The behavior of an unbraced ceiling system is similar to that of a pendulum; therefore, the lateral displacement is approximately proportional to the level of velocity-controlled ground motion and the square root of the suspension length. Therefore, a new section has been added that permits exemption from force calculations if certain displacement criteria are met. The default displacement limit has been determined based on anticipated damping and energy absorption of the suspended ceiling system assuming minimal significant impact with the perimeter wall.

6.2.7 Access Floors: Performance of computer access floors during past earthquakes and during cyclic load tests indicate that typical raised access floor systems may behave in a brittle manner and exhibit little reserve capacity beyond initial yielding or failure of critical connections. Recent testing indicates that individual panels may "pop out" of the supporting grid during seismic motions. Consideration should be given to mechanically fastening the individual panels to the supporting pedestals or stringers in egress pathways.

It is acceptable practice for systems with floor stringers to calculate the seismic force F_p for the entire access floor system within a partitioned space and then distribute the total force to the individual braces or pedestals. Stringerless systems need to be evaluated very carefully to ensure a viable seismic load path.

Overturning effects for the design of individual pedestals is a concern. Each pedestal usually is specified to carry an ultimate design vertical load greatly in excess of the W_p used in determining the seismic force F_p. It is non-conservative to use the design vertical load simultaneously with the design seismic force when considering anchor bolts, pedestal bending, and pedestal welds to base plate. The maximum concurrent vertical load when considering overturning effects is therefore limited to the W_p used in determining F_p. "Slip on" heads are not mechanically fastened to the pedestal shaft and provide doubtful capacity to transfer overturning moments from the floor panels or stringers to the pedestal.

To preclude brittle failure behavior, each element in the seismic load path must demonstrate the capacity for elastic or inelastic energy absorption. Buckling failure modes also must be prevented. Lesser seismic force requirements are deemed appropriate for access floors designed to preclude brittle and buckling failure modes.

6.2.8 Partitions: Partitions are sometimes designed to run only from floor to a suspended ceiling which provides doubtful lateral support. Partitions subject to these requirements must have independent lateral support bracing from the top of the partition to the building structure or to a substructure attached to the building structure.

6.2.9 Steel Storage Racks: Storage racks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.3.3.

6.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions: Glass performance in earthquakes can fall into one of four categories:

a. The glass remains unbroken in its frame or anchorage.

b. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier, and be otherwise serviceable.

c. The glass shatters but remains in its frame or anchorage in a precarious condition, liable to fall out at any time.

d. The glass falls out of its frame or anchorage, either in fragments, shards, or whole panels.

Categories a. and b. provide both life safety and immediate occupancy levels of performance. In the case of category b., even though the glass is cracked, it continues to provide a weather enclosure and barrier, and its replacement can be planned over a period of time. (Such glass replacement need not be performed in the immediate aftermath of the earthquake.) Categories c. and d. cannot provide for immediate occupancy, and their provision of a life safety level of performance depends on the post-breakage characteristics of the glass and the height from which it can fall. Tempered glass shatters into multiple, pebble-size fragments that fall from the frame or anchorage in clusters. These broken glass clusters are relatively harmless to humans when they fall from limited heights, but when they fall from greater heights they could be harmful.

6.2.10.1 General: Eq. 6.2.10.1-2 is derived from *Earthquake Safety Design of Windows*, published in November 1982 by the Sheet Glass Association of Japan. Eq. 6.2.10.1-2 is derived from a similar equation in Bouwkamp and Meehan (1960) that permits calculation of the interstory drift required to cause glass-to-frame contact in a given rectangular window frame. Both equations are based on the principle that a rectangular window frame (specifically, one that is anchored mechanically to adjacent stories of the primary structural system of the building) becomes a parallelogram as a result of interstory drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself.

The 1.25 factor in Eqs. 6.2.10.1-1 and 6.2.10.1-2 reflect uncertainties associated with calculated inelastic seismic displacements in building structures. Wright (1989) stated that "post-elastic deformations, calculated using the structural analysis process, may well underestimate the actual building deformation by up to 30 percent. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum interstory displacement to verify adequate performance." Therefore, Wright's comments form the basis for employing the 1.25 factor in Eqs. 6.2.10.1-1 and 6.2.10.1-2.

6.2.10.2 Seismic Drift Limits for Glass Components

Introduction

Seismic design requirements for glass in building codes have traditionally been non-existent or limited to the general statement that "drift be accommodated." No distinction has been made regarding the seismic performance of different types of glass, different frames, and different glazing systems. Yet, significant differences exist in the performance of various glass types subjected to simulated earthquake conditions. Controlled laboratory studies were conducted to investigate the cracking resistance and fallout resistance of different types of glass installed in the same storefront and mid-rise wall systems. Effects of glass surface prestress, lamination, wall system type, and dry versus structural silicone glazing were considered. Laboratory results revealed that distinct magnitudes of interstory drift cause glass cracking and glass fallout in each glass type tested. Notable differences in seismic resistance exist between glass types commonly used in contemporary building design.

Test Facility and Experimental Plan

In-plane dynamic racking tests were performed using the facility shown in Figure C6.2.10.2-1. Rectangular steel tubes at the top and bottom of the facility are supported on roller assemblies, which permit only horizontal motion of the tubes. The bottom steel tube is driven by a computer-controlled hydraulic ram, while the top tube is attached to the bottom tube by means of a fulcrum and pivot arm assembly. This mechanism causes the upper steel tube to displace the same amount as the lower steel tube, but in the opposite direction, which doubles the amount of interstory drift that can be imposed on a test specimen from ± 76 mm (± 3 in.) to ± 152 mm (± 6 in.). The test facility accommodated up to three glass test panels, each 1.5 m (5 ft) wide x 1.8 m (6 ft) high. A more detailed description of the dynamic racking test facility is included in Behr and Belarbi (1996).

Several types of glass, shown in Table C6.2.10.2-1, were tested under simulated seismic conditions in the storefront and mid-rise dynamic racking tests. These glass types, along with the wall systems employed in the tests, were selected after polling industry practitioners and wall system designers for their opinions regarding common glass types and common wall system types employed in contemporary storefront and mid-rise wall constructions.

Storefront Wall System Tests

Tests were conducted on various glass types dry-glazed within a wall system commonly used in storefront applications. Loading histories for the storefront wall system tests were based on dynamic analyses performed on a "typical" storefront building that was not designed specifically for seismic resistance (Pantelides et al., 1996). Two types of tests were conducted on the storefront wall systems: (1) serviceability tests, wherein the drift loading history of the glass simulated the response of a storefront building structure to a "maximum probable" earthquake event; and (2) ultimate tests, wherein drift amplitudes were twice those of the serviceability tests, which was a simplified means of approximating the loading history of a "maximum credible" earthquake event. As indicated in Table C6.2.10.2-1, five glass types were tested, all dry-glazed in a storefront wall system. Three glass panels were mounted side by side in the test facility, after which horizontal (in-plane) racking motions were applied.

TABLE C6.2.10.2-1. - GLASS TYPES INCLUDED IN STOREFRONT AND MID-RISE DYNAMIC RACKING TESTS

GLASS TYPE	Storefront Tests	Mid-Rise Tests
6 mm (1/4 in.) Annealed Monolithic	✓	✓
6 mm (1/4 in.) Heat-Strengthened Monolithic		✓
6 mm (1/4 in.) Fully Tempered Monolithic	✓	✓
6 mm (1/4 in.) Annealed Monolithic with 0.1 mm PET Film (film not anchored to wall system frame)		✓
6 mm (1/4 in.) Annealed Laminated	✓	✓
6 mm (1/4 in.) Heat-Strengthened Laminated		✓
6 mm (1/4 in.) Heat-Strengthened Monolithic Spandrel		✓
25 mm (1 in.) Annealed Insulating Glass Units	✓	✓
25 mm (1 in.) Heat-Strengthened Insulating Glass Units		✓

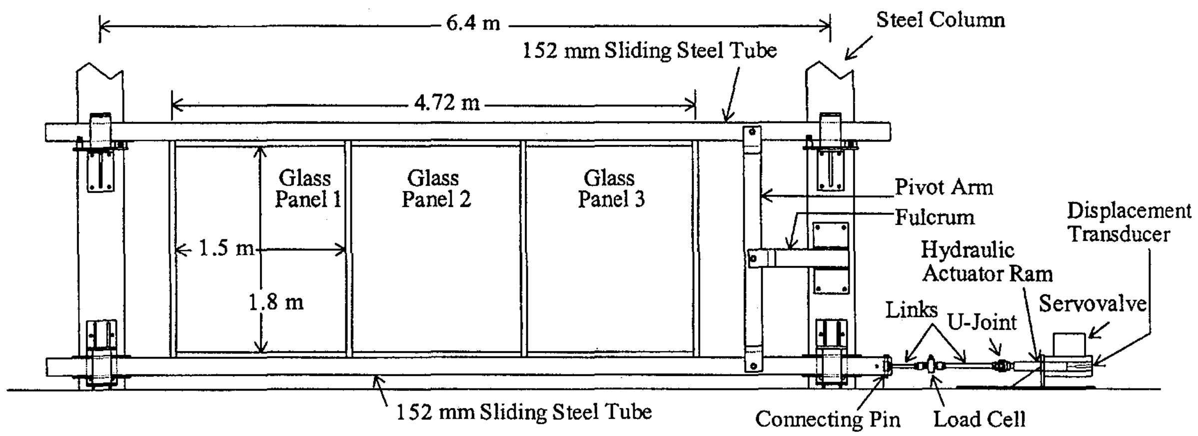


Figure C6.2.10.2 - 1 - Dynamic Racking Test Facility (1 m = 3.28 ft)

FIGURE C6.2.10.2-1 Dynamic racking test Facility.

The serviceability test lasted approximately 55 seconds and incorporated drift amplitudes ranging from ± 6 to ± 44 mm (± 0.25 to ± 1.75 in.). The drift pattern in the ultimate test was formed by doubling each drift amplitude in the serviceability test. Both tests were performed at a nominal frequency of 0.8 Hz.

Experimental results indicated that for all glass types tested, serviceability limit states associated with glass edge damage and gasket seal degradation in the storefront wall system were exceeded during the moderate earthquake simulation (i.e., the serviceability test). Ultimate limit states associated with major cracking and glass fallout were reached for the most common storefront glass type, 6 mm (1/4 in.) annealed monolithic glass, during the severe earthquake simulation (i.e., the ultimate test). This observation is consistent with a reconnaissance report of damage resulting from the Northridge Earthquake (EERI, 1994). More information regarding the storefront wall system tests is included in Behr, Belarbi and Brown (1995). In addition to the serviceability and ultimate tests, increasing-amplitude "crescendo tests," similar to those described below for the mid-rise tests, were performed at a frequency of 0.8 Hz on selected storefront glass types. Results of these crescendo tests are reported in Behr, Belarbi and Brown (1995) and are included in some of the comparisons made below.

Mid-Rise Curtain Wall System Tests

Another series of tests focused on the behavior of glass panels in a popular curtain wall system for mid-rise buildings. All mid-rise glass types in Table C6.2.10.2-1 were tested with a dry-glazed wall system that uses polymeric (rubber) gaskets wedged between the glass edges and the curtain wall frame to secure each glass panel perimeter. In addition, three glass types were tested with a bead of structural silicone sealant on the vertical glass edges and dry glazing gaskets on the horizontal edges (i.e., a "two-side structural silicone glazing system"). Six specimens of each glass type were tested.

Crescendo tests were performed on all mid-rise test specimens. As described by Behr and Belarbi (1996), the crescendo test consisted of a series of alternating "ramp-up" and "constant amplitude" intervals, each containing four, sinusoidal-shaped drift cycles. Each drift amplitude "step" (i.e., the increase in amplitude between adjacent constant amplitude intervals, which was achieved by completing the four cycles in the intermediary ramp-up interval) was ± 6 mm (± 0.25 in.). The entire crescendo test sequence lasted approximately 230 seconds. Crescendo tests on mid-rise glass specimens were conducted at 1.0 Hz for dynamic racking amplitudes from 0 to 114 mm (0 to 4.5 in.), 0.8 Hz for amplitudes from 114 to 140 mm (4.5 to 5.5 in.), and 0.5 Hz for amplitudes from 140 to 152 mm (5.5 to 6 in.). These frequency reductions at higher racking amplitudes were necessary to avoid exceeding the capacity of the hydraulic actuator ram in the dynamic racking test facility.

The drift magnitude at which glass cracking was first observed was called the "serviceability drift limit," which corresponds to the drift magnitude at which glass damage would necessitate glass replacement. The drift magnitude at which glass fallout occurred was called the "ultimate drift limit," which corresponds to the drift magnitude at which glass damage would become a life safety hazard. This ultimate drift limit for architectural glass is related to "$\Delta_{fallout}$" in Sec. 6.2.10.1 of the *Provisions,* noting that horizontal racking displacements (i.e., drifts) in the crescendo tests were typically applied to test specimens having panel heights of only 1.8 m (6 ft).

In addition to recording the serviceability drift limit and ultimate drift limit for each glass test specimen, the drift magnitude causing first contact between the glass panel and the aluminum frame was also recorded. To establish when this contact occurred, thin copper wires were attached to each corner of the glass panel and were connected to an electronics box. If the copper wire came into contact with the aluminum frame, an indicator light on an electronics box was actuated. Measured drifts causing glass-to-aluminum contact correlated well with those predicted by Eq. 6.2.10.1-2.

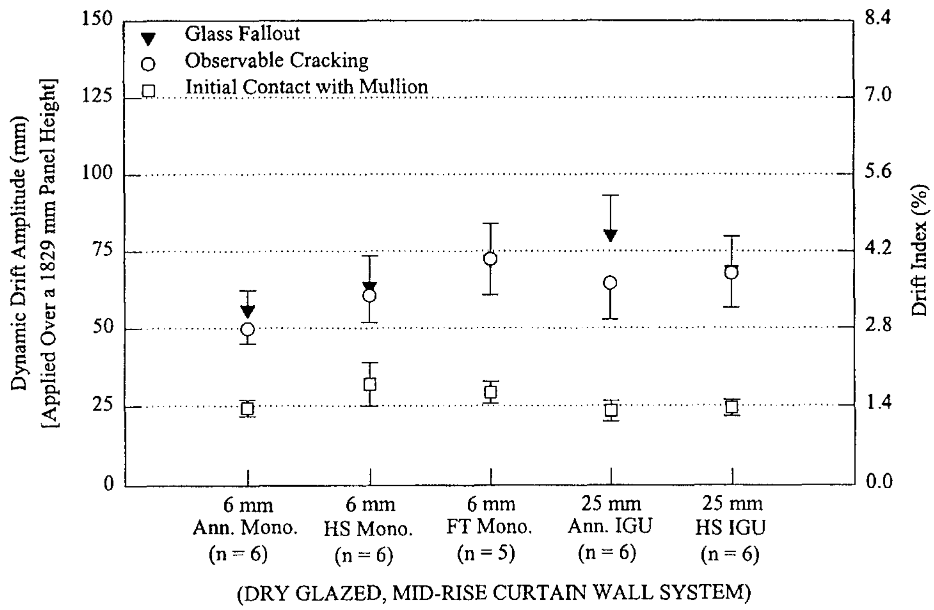
Glass Failure Patterns From Crescendo Tests

Glass failure patterns were recorded during each storefront test and mid-rise test. Annealed monolithic glass tended to fracture into sizeable shards, which then fell from the curtain wall frame. Heat-strengthened monolithic glass generally broke into smaller shards than annealed monolithic glass, with the average shard size being inversely proportional to the magnitude of surface compressive prestress in the glass. Fully tempered monolithic glass shattered into much smaller, cube-shaped fragments. Annealed monolithic glass with unanchored 0.1 mm (4 mil) PET film also fractured into large shards, much like un-filmed annealed monolithic glass, but the shards adhered to the film. However, when the weight of the glass shards became excessive, the entire shard/film conglomeration sometimes fell from the glazing pocket as a unit. Thus, unanchored 0.1 mm PET film was not observed to be totally effective in terms of preventing glass fallout under simulated seismic loadings, which agrees with field observations made in the aftermath of the 1994 Northridge Earthquake (Gates and McGavin, 1998). Annealed and heat-strengthened laminated glass units experienced fracture on each glass ply separately, which permitted these laminated glass units to retain sufficient rigidity to remain in the glazing pocket after one (or even both), glass plies had fractured due to glass-to-aluminum contacts. Annealed and heat-strengthened laminated glass units exhibited the highest resistance to glass fallout during the dynamic racking tests.

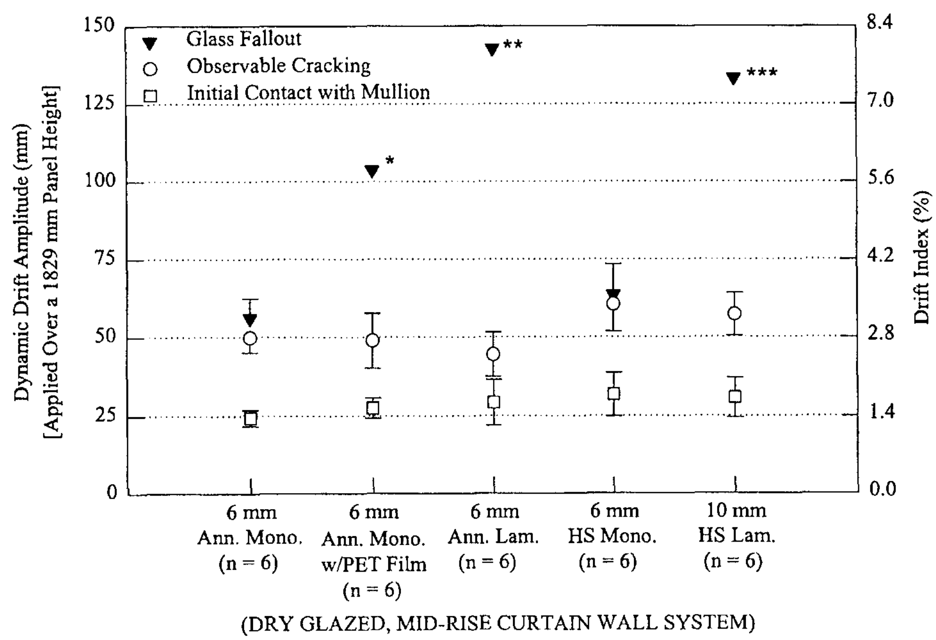
Quantitative Drift Limit Data From Crescendo Tests

Serviceability and ultimate drift limit data obtained during the crescendo tests are presented in four windows in Figure C6.2.10.2-2. Figure C6.2.10.2-2a shows the effects of glass surface prestress (i.e., annealed, heat-strengthened and fully tempered glass) on seismic drift limits; Figure C6.2.10.2-2b shows the effects of lamination (i.e., monolithic glass, monolithic glass with unanchored 0.1 mm PET film, and laminated glass); Figure C6.2.10.2-2c shows the effects of wall system type (i.e., lighter, more flexible, storefront wall system versus the same glass types tested in a heavier, stiffer, mid-rise wall system); and Figure C6.2.10.2-2d shows the effects of structural silicone glazing (i.e., dry glazing versus two-side structural silicone glazing). Each symbol plotted in Figure C6.2.10.2-2 is the mean value for specimens of a given glass type, along with ± one standard deviation error bars. In those cases where error bars for a particular glass type overlap, only one side of the error bar is plotted. In cases where the glass panel did not experience fallout by the end of the crescendo test, a conservative ultimate drift limit magnitude of 152 mm (6 in.) (the racking limit of the test facility) is assigned for plotting purposes in Figure C6.2.10.2-2. (This ultimate drift limit, shown with a "▾" symbol in Figure C6.2.10.2-2, is related to the term "$\Delta_{fallout}$" in Sec. 6.2.10.1 of the *Provisions.*) No error bars are plotted for these "pseudo data points," since the drift magnitude at which the glass panel would actually have experienced fallout could not be observed; certainly, the actual ultimate drift limits for these specimens are greater than ±152 mm (± 6 in.).

The ±152 mm (± 6 in.) racking limit of the test facility, when applied over the 1829 mm (72 in.) height of glazing panel specimens represents a severe interstory drift index of over 8 percent. This 8 percent drift index exceeds, by a significant margin, provisions in Sec. 5.2.8 (Table 5.2.8) that set allowable drift limits between 0.7 percent and 2.5 percent, depending on structure type and Seismic Use Group. Thus, the drift limits, Δ_a, in Table 5.2.8 are considerably lower than the racking limits of the laboratory facility used for the crescendo tests. In building design, however, values of $\Delta_{fallout}$ would need to be significantly higher than the interstory drifts exhibited by the primary building structure in order to provide an acceptable safety margin against glass fallout.



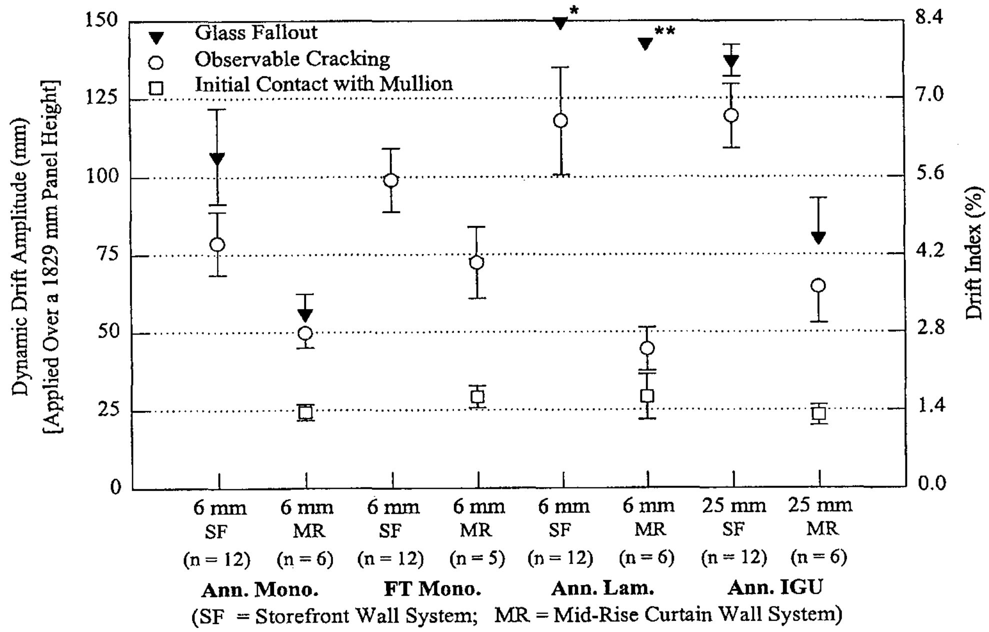
(a) Effects of Glass Surface Prestress



*1of 6 specimens did not fall out. **5 of 6 specimens did not fall out. ***2 of 6 specimens did not fall out.

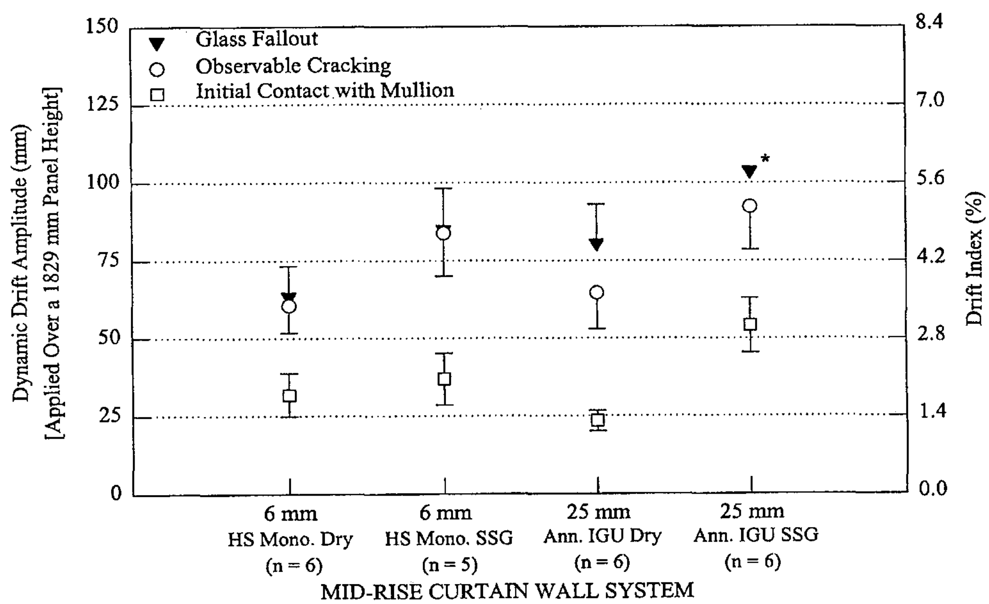
(b) Effects of Lamination

FIGURE C6.2.10.2 - 2 - Seismic Drift Limits from Crescendo Tests on Architectural Glass



*8 of 12 specimens did not fall out. **5 of 6 specimens did not fall out.

(c) Effects of Wall System Type



*1 of 6 specimens did not fall out.

(d) Effects of Structural Silicone Glazing

FIGURE C6.2.10.2 - 2 (continued) - Seismic Drift Limits from Crescendo Tests on Architectural Glass

Summary Observations From Figure C6.2.10.2-2:

(a) Effects of Glass Surface Prestress - Figure C6.2.10.2-2a illustrates the effects of glass surface prestress on observed seismic drift limits. To eliminate all variables except for glass surface prestress, data from only the mid-rise curtain wall tests are plotted. Slight increases in cracking and fallout drift limits can be seen for 6 mm (0.25 in.) monolithic glass as the level of glass surface prestress increases from annealed to heat-strengthened to fully tempered glass. However, effects of glass surface prestress on observed seismic drift limits were statistically significant only when comparing 6 mm fully tempered monolithic glass to 6 mm annealed monolithic glass. All six of the 6 mm fully tempered monolithic glass specimens shattered when initial cracking occurred, causing the entire glass panels to fall out. Similar behavior was observed in four of the six 6 mm heat-strengthened monolithic glass specimens. No appreciable differences in seismic drift limits existed between annealed and heat-strengthened 25 mm (1 in.) insulating glass units.

(b) Effects of Lamination - Figure C6.2.10.2-2b shows the effects of lamination configuration on seismic drift limits. Lamination had no appreciable effect on the drift magnitudes associated with first observable glass cracking. In a dry-glazed system, the base glass type (and not the lamination configuration) appeared to control the drift magnitude associated with glass cracking. However, lamination configuration had a pronounced effect on glass fallout resistance (i.e., $\Delta_{fallout}$). Specifically, monolithic glass types were more prone to glass fallout than were either annealed monolithic glass with unanchored 0.1 mm PET film or annealed laminated glass. All six annealed monolithic glass panels experienced glass fallout during the tests; five of six annealed monolithic glass specimens with unanchored 0.1 mm PET film experienced fallout; only one of six annealed laminated glass panels experienced fallout.

Laboratory tests also showed that heat-strengthened laminated glass had higher fallout resistance than did heat-strengthened monolithic glass. Heat-strengthened monolithic glass panels fell out at significantly lower drift magnitudes than did heat-strengthened laminated glass units. Heat-strengthened laminated glass units tended to fall out in one large piece, instead of exhibiting the smaller shard fallout behavior of heat-strengthened monolithic glass.

(c) Effects of Wall System Type - Figure C6.2.10.2-2c illustrates the effects of wall system type on observed seismic drift limits. For all four glass types tested in both the storefront and mid-rise wall systems, the lighter, more flexible storefront frames allowed larger drift magnitudes before glass cracking or glass fallout than did the heavier, stiffer mid-rise curtain wall frames. This observation held true for all glass types tested in both wall system types.

(d) Effects of Structural Silicone Glazing - As shown in Figure C6.2.10.2-2d, use of a two-side structural silicone glazing system increased the dynamic drift magnitudes associated with first observable glass cracking in both heat-strengthened monolithic glass and annealed insulating glass units. During the crescendo tests, glass panels were observed to "walk" horizontally across the frame after the beads of structural silicone sealant had sheared. Because the mid-rise curtain wall crescendo tests were performed on single glass panels, the glass specimen was unobstructed as it walked horizontally across the frame. In a multi-panel curtain wall assembly on an actual building, adjacent glass panels could collide, which could induce glass cracking at lower drift magnitudes than those observed in the single-panel tests performed in this study. It is also clear from Figure C6.2.10.2-2d that glass specimens with two-side structural silicone glazing exhibited higher resistance to glass fallout than did comparable dry-glazed glass specimens.

Conclusion

Dynamic racking tests showed that distinct and repeatable dynamic drift magnitudes were associated with glass cracking and glass fallout in various types of glass tested in storefront and mid-rise wall systems. Seismic resistance varied widely between glass types commonly employed in contemporary building design. Annealed and heat-strengthened laminated glass types exhibited higher resistance to glass fallout than did monolithic glass types. Annealed monolithic glass with unanchored 0.1 mm PET film exhibited total fallout of the glass shard/adhesive film conglomeration in five out of six of the crescendo tests performed.

Glass panels glazed within stiffer aluminum frames were less tolerant of glass-to-aluminum collisions and were associated with glass fallout events at lower drift magnitudes than were the same glass types tested in a more flexible aluminum frame. Glazing details were also found to have significant effects on the seismic performance of architectural glass. Specifically, architectural glass within a wall system using a structural silicone glaze on two sides exhibited higher seismic resistance than did identical glass specimens dry-glazed on all four sides within a comparable wall framing system.

References

Behr, R.A., Belarbi, A. and Brown, A.T., 1995, "Seismic Performance of Architectural Glass in a Storefront Wall System," *Earthquake Spectra*, Vol. 11, No. 3, pp. 367-391.

Behr, R.A. and Belarbi, A., 1996, "Seismic Tests Methods for Architectural Glazing Systems," *Earthquake Spectra*, Vol. 12, No. 1, pp. 129-143.

Earthquake Engineering Research Institute (EERI), 1994, "Northridge Earthquake January 17, 1994: Preliminary Reconnaissance Report," ed. John F. Hall, Oakland, California, pp. 56-57.

Gates, W.E. and McGavin, G., 1998, "Lessons Learned from the 1994 Northridge Earthquake on the Vulnerability of Nonstructural Systems," Proceedings, Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components, ATC-29-1, January 22-23, 1998, San Francisco, California, Applied Technology Council, Redwood City, California, 93-101.

Pantelides, C.P., Truman, K.Z., Behr, R.A. and Belarbi, A., 1996, "Development of a Loading History for Seismic Testing of Architectural Glass in a Shop-Front Wall System," *Engineering Structures*, Vol. 18, No. 12, pp. 917-935.

Acknowledgment

The National Science Foundation (Grant No. CMS 9213172) provided major funding for the experimental results summarized in Sec. C6.2.10.2.

End Note: The American Architectural Manufacturers Association (AAMA) has issued AAMA 501.4-2000: "Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Interstory Drifts." In contrast with the dynamic displacements employed in the crescendo tests described in this section, static displacements are employed in AAMA's recommended test method. Correlations between the results of the static and dynamic test methods have not yet been established with regard to the seismic performance of architectural glazing systems.

6.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN:

6.3.1 General: The primary focus of these requirements is on the design of attachments and equipment supports for mechanical and electrical *components*.

The requirements are intended to reduce the hazard to life posed by the loss of *component* structural stability or integrity. The requirements should increase the reliability of *component* operation but do not directly address the assurance of functionality.

The design of mechanical and electrical *components* must consider two levels of earthquake safety. For the first safety level, failure of the mechanical or electrical *component* itself poses no significant hazard. In this case, the only hazard posed by the *component* is if the support and the means by which the *component* and its supports are attached to the building or the ground fails and the *component* could slide, topple, fall, or otherwise move in a manner that creates a hazard for persons nearby. In the first category, the intent of these requirements is only to design the support and the means by which the *component* is attached to the structure, defined in the Glossary as "equipment supports" and "attachments." For the second safety level, failure of the mechanical or electrical equipment itself poses a significant hazard. In this case, failure could either be to a containment having hazardous contents or contents required after the earthquake or failure could be functional to a *component* required to remain operable after an earthquake. In this second category, the intent of these requirements is to provide guidance for the design of the *component* as well as the means by which the *component* is supported and attached to the structure. The requirements should increase the survivability of this second category of *component* but the assurance of functionality may require additional considerations. Examples of this second category include fire protection piping or an uninterruptible power supply in a hospital. Another example involves the rupture of a vessel or piping that contains sufficient quantities of highly toxic or explosive substances such that a release would be hazardous to the safety of building occupants or the general public. In assessing whether failure of the mechanical or electrical equipment itself poses a hazard, certain judgments may be necessary. For example, small flat-bottom tanks themselves may not need to be designed for earthquake loads; however, numerous seismic failures of large flat-bottom tanks and the hazard of a large fluid spill suggest that many, if not most, of these should be. Distinguishing between large and small, in this case, may require an assessment of potential damage caused by a spill of the fluid contents over and above the guidance offered in Sec. 6.3.9.

It is intended that the requirements provide guidance for the design of *components* for both conditions in the second category. This is primarily accomplished by increasing the design forces with an importance factor, I_p. However, this only affects structural integrity and stability directly. Function and operability of mechanical and electrical *components* may only indirectly be affected by increasing design forces. For complex *components*, testing or experience may be the only reasonable way to improve the assurance of function and operability. On the basis of past earthquake experience, it may be concluded that if structural integrity and stability are maintained, function and operability after an earthquake will be reasonably provided for most types of equipment *components*. On the other hand, mechanical joints in containment *components* (tanks, vessels, piping, etc.) may not remain leaktight in an earthquake even if after the earthquake leaktightness is re-established. Judgment may suggest a more conservative design related in some manner to the perceived hazard than would otherwise be provided by these requirements.

It is not intended that all equipment or parts of equipment be designed for seismic forces. Determination of whether these requirements need to be applied to the design of a specific piece of equipment or a part of that equipment will sometimes be a difficult task. Damage to or even failure of a piece or part of a *component* is not a concern of these requirements so long as a hazard to life does not exist. Therefore, the restraint or containment of a falling, breaking, or toppling *component* or its parts by the use of bumpers, braces, guys, wedges, shims, tethers, or gapped restraints often may be an acceptable approach to satisfying these requirements even though the *component* itself may suffer damage. Judgment will be required if the intent of these requirements is to be fulfilled. The following example may be helpful: Since the threat to life is a key consideration, it should be clear that a nonessential air handler package unit that is less than 4 ft (1.2 m) tall bolted to a mechanical room floor is not a threat to life as long as it is prevented from significant motions by having adequate anchorage. Therefore, earthquake design of the air handler itself need not be performed. However, most engineers would agree that a 10-ft (3.0 m) tall tank on 6-ft (1.8 m) angles used as legs mounted on the roof near a building exit does pose a hazard. It is the intent of these requirements that the tank legs, the connections between the roof and the legs, the connections between the legs and the tank, and possibly even the tank itself be designed to resist earthquake forces. Alternatively, restraint of the tank by guys or bracing could be acceptable.

It is not the intent of the *Provisions* to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. When the potential for a hazard to life exists, it is expected that design efforts will focus on equipment supports including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, or ties.

Many mechanical and electrical *components* consist of complex assemblies of mechanical and/or electrical parts that typically are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and often are designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it may be inherently rugged. Rugged, as used herein, refers to an ampleness of construction that renders such equipment the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional usually should be able to confirm such ruggedness. The results of equipment ruggedness assessment then will determine the need for an appropriate method and extent of the seismic design or qualification efforts.

It also is recognized that a number of professional and industrial organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical *components*. In addition to providing design guidance for normal and upset operating conditions and various environmental conditions, some have developed earthquake design guidance in the context of the overall mechanical or electrical design. It is the intent of these requirements that such codes and standards having earthquake design guidance be used as it is to be expected that the developers have a greater familiarity with the expected failure modes of the *components* for which their design and construction rules are developed. In addition, even if such codes and standards do not have earthquake design guidance, it is generally regarded that construction of mechanical and electrical equipment to nationally recognized codes and standards such as those approved by the American National Standards Institute provide adequate strength (with a safety margin often greater than that provided by structural codes) to accommodate all normal and upset operating loads. In this case, it could also be assumed that the *component* has sufficient strength (especially if constructed of ductile materials) to not break up or break away from its supports in such a way as to provide a life-safety hazard. Earthquake damage surveys confirm this.

Specific guidance for selected *components* or conditions is provided in Sec. 6.3.6 through 6.3.16.

6.3.2 Mechanical and Electrical Component Forces and Displacements: *Components* that could be damaged or could damage other *components* and are fastened to multiple locations of a structure should be designed to accommodate seismic relative displacements. Examples of *components* that should be designed to accommodate seismic relative displacements include bus ducts, cable trays, conduit, elevator guide rails, and piping systems.

6.3.3 Mechanical and Electrical Component Period: Determination of the fundamental period of an item of mechanical or electrical equipment using analytical or in-situ testing methods can become very involved and can produce nonconservative results (i.e., underestimated fundamental periods) if not properly performed.

When using analytical methods, it is absolutely essential to define in detail the flexibility of the elements of the equipment base, load path, and attachment to determine K_p. This base flexibility typically dominates equipment *component* flexibility and thus fundamental period.

When using test methods, it is necessary to ensure that the dominant mode of vibration of concern for seismic evaluation is excited and captured by the testing. This dominant mode of vibration typically cannot be discovered in equipment in-situ tests that measure only ambient vibrations. In order for the highest fundamental period dominant mode of vibration to be excited by in-situ tests, relatively significant input levels of motion are required (i.e., the flexibility of the base and attachment needs to be exercised).

Many types of mechanical equipment *components* have fundamental periods below 0.06 sec and may be considered to be rigid. Examples include horizontal pumps, engine generators, motor generators, air compressors, and motor driven centrifugal blowers. Other types of mechanical equipment also are very stiff but may have fundamental periods up to approximately 0.125 sec. Examples of these mechanical equipment items include vertical immersion and deep well pumps, belt driven and vane axial fans, heaters, air handlers, chillers, boilers, heat exchangers, filters, and evaporators. These fundamental period estimates do not apply when the equipment is on vibration-isolator supports.

Electrical equipment cabinets can have fundamental periods of approximately 0.06 to 0.3 sec depending upon weight, stiffness of the enclosure assembly, flexibility of the enclosure base, and load path through to the attachment points. Tall and narrow motor control centers and switchboards lie in the upper end of this period range. Low and medium-voltage switchgear, transformers, battery chargers, inverters, instrumentation cabinets, and instrumentation racks usually have fundamental periods ranging from 0.1 to 0.2 sec. Braced battery racks, stiffened vertical control panels, benchboards, electrical cabinets with top bracing, and wall-mounted panelboards have fundamental periods ranging from 0.06 to 0.1 sec.

6.3.4 Mechanical and Electrical Component Attachments: For some items such as piping, relative seismic displacements between support points generally are of more significance than inertial forces. *Components* made of ductile materials such as steel or copper can accommodate relative displacement effects by inelastically conforming to the supports' conditions. However, *components* made of less ductile materials can only accommodate relative displacement effects by providing flexibility or flexible connections.

Of most concern are distribution systems that are a significant life-safety hazard and are routed between two separate building structures. Ductile *components* with bends and elbows at the building separation point or *components* that will be subject to bending stresses rather than direct tensile loads due to differential support motion, are not so prone to damage and are not so likely to fracture and fall. This is valid if the supports can accommodate the imposed loads.

6.3.5 Component Supports: It is the intent of these requirements to ensure that all mechanical and electrical *component* supports, the means by which a *component* transfers seismic loads to the structure, be designed to accommodate the force and displacement effects prescribed. *Component* supports are differentiated here from *component* attachments to emphasize that the supports themselves, the structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, even if fabricated with and/or by the mechanical or electrical *component* manufacturer, should be designed for seismic forces. This is regardless of whether the mechanical or electrical *component* itself is designed for seismic loads. The intention is to prevent a *component* from sliding, falling, toppling, or otherwise moving such that the *component* would imperil life.

6.3.6 Component Certification: It is intended that the certificate only be requested for *components* with an importance factor (I_p) greater than 1.00 and only if the *component* has a doubtful or uncertain seismic load path. This certificate should not be requested to validate functionality concerns.

In the context of the *Provisions*, seismic adequacy of the *component* is of concern only when the *component* is required to remain operational after an earthquake or contains material that can pose a significant hazard if released. Meeting the requirements of this section shall be considered as an acceptable demonstration of the seismic adequacy of a *component*.

6.3.7 Utility and Service Lines at Structure Interfaces: For essential facilities, auxiliary on-site mechanical and electrical utility sources are recommended. It is recommended that an appropriate clause be included if existing codes for the jurisdiction do not presently provide for it.

Sec. 6.3.7 requires that adequate flexibility be provided for utilities at the interface of adjacent and independent structures to accommodate anticipated differential displacement. It affects architectural and mechanical/electrical fittings only where water and energy lines pass through the interface. The displacements considered must include the C_d factor of Sec. 5.2.2 and should be in accordance with *Provisions* Sec. 6.1.4.

Consideration may be necessary for nonessential piping carrying quantities of materials that could, if the piping is ruptured, damage essential utilities.

Following a review of information from the Northridge and Loma Prieta earthquakes and discussions with gas company personnel, automatic earthquake shutoff of gas lines at structure entry points is no longer required. The primary justification for this is the consensus opinion that shutoff devices tend to cause more problems than they solve. Commercially available shutoff devices tend to be susceptible to inadvertent shutoff caused by passing vehicles and other non-seismic vibrations. This leads to disruption of service and often requires that local gas companies reset the device and relight any pilot lights. In an earthquake, the majority of shutoff devices which actuate will be attached to undamaged gas lines. This results in a huge relight effort for the local utility at a time when resources are typically at a premium. If the earthquake occurs during the winter, a greater life hazard may exist from a lack of gas supply than from potential gas leaks. In the future, as shutoff devices improve and gas-fired appliances which use pilots are phased out, it may be justified to require shutoff devices.

This is not meant to discourage individuals and companies from installing shutoff devices. In particular, individuals and companies who are capable of relighting gas fired equipment should seriously consider installation of these devices. In addition, gas valves should be closed whenever leaks are detected.

6.3.9 Storage Tanks: Storage tanks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.7.3.

6.3.10 HVAC Ductwork: Experience in past earthquakes has shown that, in general, HVAC duct systems are rugged and perform well in strong shaking motions. Bracing in accordance with the Sheet Metal and Air Conditioning Contractors National Association SMACNA HVAC, SMACNA Rectangular, SMACNA Restraint has been shown to be effective in limiting damage to duct systems under earthquake loads. Typical failures have affected system function only and major damage or collapse has been uncommon. Therefore, industry standard practices should prove adequate for most installations. Expected earthquake damage should be limited to opening of the duct joints and tears in the ducts. Connection details that are prone to brittle failures, especially hanger rods subject to large amplitude bending stress cycles, should be avoided. Some ductwork systems carry hazardous materials or must remain operational during and after an earthquake. These ductwork system would be designated as having an I_p greater than 1.0. A detailed engineering analysis for these systems should be performed.

All equipment (e.g., fans, humidifiers, and heat exchangers) attached to the ducts and weighing more than 75 lb (334 N) should be braced independently of the duct. Unbraced in-line equipment can damage the duct by swinging and impacting it during an earthquake. Items (e.g., dampers, louvers, and air diffusers) attached to the duct should be positively supported by mechanical fasteners (not friction-type connections) to prevent their falling during an earthquake. Where it is desirable to limit the deflection of duct systems under seismic load, bracing in accordance with the SMACNA references listed in Sec. 6.1.1 may be used.

6.3.11 Piping Systems: Experience in past earthquakes has shown that, in general, piping systems are rugged and perform well in strong shaking motions. Numerous standards and guidelines have been developed covering a wide variety of piping systems and materials. Construction in accordance with current requirements of the referenced national standard shave been shown to be effective in limiting damage to and avoiding loss of fluid containment in piping systems under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized standards be used to design piping systems provided that the force and displacement demand is equal to or exceeds the requirements of Sec. 6.1.3 and 6.1.4 and provisions are made to mitigate seismic interaction issues not normally addressed in the national standards. The following industry standards, while not adopted by ANSI, are in common use and may be appropriate reference documents for use in the seismic design of piping systems. SMACNA *Guidelines for the Seismic Restraint of Mechanical Systems* ASHRAE CH 50-95 *Seismic Restraint Design Piping*, as used herein, are assemblies of pipe, tubing, valves, fittings, and other in-line fluid containing *components*, excluding their attachments and supports.

6.3.12 Boilers and Pressure Vessels: Experience in past earthquakes has shown that, in general, boilers and pressure vessels are rugged and perform well in strong shaking motions. Construction in accordance with current requirements of the ASME *Boiler and Pressure Vessel Code* (ASME/BPV) has been shown to be effective in limiting damage to and avoiding loss of fluid containment in boilers and pressure vessels under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized codes be used to design boilers and pressure vessels provided that the seismic force and displacement demand is equal to or exceeds the requirements of Sec. 6.1.3 and 6.1.4. Until such nationally recognized codes incorporate force and displacement requirements comparable to the requirements of Sec. 6.1.3 and 6.1.4, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

Boilers and pressure vessels as used herein are fired or unfired containments, including their internal and external appurtenances and internal assemblies of pipe, tubing, and fittings, and other fluid containing *components*, excluding their attachments and supports.

6.3.13 Mechanical Equipment Attachments and Supports: Past earthquakes have demonstrated that most mechanical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of mechanical equipment items for operational and transportation loads typically envelopes loads due to earthquake. For this reason, the requirements primarily focus on equipment anchorage and attachments. It was felt, however, that mechanical equipment *components* required to maintain containment of flammable or hazardous materials should themselves be designed for seismic forces.
In addition, there liability of equipment operability after an earthquake can be increased if the following items are also considered in design:

a. Internal assemblies are attached with a sufficiency that eliminates the potential of impact with other internal assemblies and the equipment wall; and
b. Operators, motors, generators, and other such *components* functionally attached mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft will be avoided.

6.3.14 Electrical Equipment Attachments and Supports: Past earthquakes have demonstrated that most electrical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of electrical equipment items for operational and transportation loads typically envelopes loads due to earthquake. For this reason, the requirements primarily focus on equipment anchorage and attachments. However, reliability of equipment operability after an earthquake can be increased if the following items also are considered in design:

a. Internal assemblies are attached with a sufficiency that electrical subassemblies and contacts will not be subject to differential movement or impact between the assemblies, contacts, and the equipment enclosure.
b. Any ceramic or other nonductile *components* in the seismic load path should be specifically evaluated.
c. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from banging into adjacent structural members.

6.3.15 Alternate Seismic Qualification Methods: Testing is a well established alternative method of seismic qualification for small to medium size equipment. Several national standards, other than IEEE 344 (IEEE-344), have testing requirements adaptable for seismic qualification.

6.3.16 Elevator Design Requirements: The ASME *Safety Code for Elevators and Escalators* (ASME A17.1) has adopted many requirements to improve the seismic response of elevators; however, they do not apply to some regions covered by this chapter. These changes are to extend force requirements for elevators to be consistent with the *Provisions*.

6.3.16.2 Elevator Machinery and Controller Supports and Attachments: The ASME *Safety Code for Elevators and Escalators* (ASME A17.1) has no seismic requirements for supports and attachments for some structures and zones where the *Provisions* are applicable. Criteria are provided to extend force requirements for elevators to be consistent with the intent and scope of the *Provisions*.

6.3.16.3 Seismic Controls: The purpose of the seismic switch as used here is different from that provided under the ASME *Safety Code for Elevators and Escalators* (ASTM C635), which has incorporated several requirements to improve the seismic response of elevators (e.g., rope snag point guard, rope retainer guards, guide rail brackets) that do not apply to some buildings and zones covered by the *Provisions*. Building motions that are expected in these uncovered seismic zones are sufficiently large to impair the operation of elevators. The seismic switch is positioned high in the structure where structural response will be the most severe. The seismic switch trigger level is set to shut down the elevator when structural motions are expected to impair elevator operations.

Elevators in which the seismic switch and counterweight derail device have triggered should not be put back into service without a complete inspection. However, in the case where the loss of use of the elevator creates a life-safety hazard, an attempt to put the elevator back into service may be attempted. Operating the elevator prior to inspection may cause severe damage to the elevator or its *components*.

The building owner should have detailed written procedures in place directing the elevator operator/maintenance personnel which elevators in the facility are necessary from a post-earthquake life safety perspective. It is highly recommended that these procedures be in-place, with appropriate personnel training prior to an event strong enough to trip the seismic switch.

Once the elevator seismic switch is reset, it will respond to any call at any floor. It is important that the detailed procedure include the posting of "out-of-service for testing" signs at each door at each floor, prior to resetting the switch. Once the testing is completed, and the elevator operator/maintenance personnel are satisfied that the elevator is safe to operate, the signs can be removed.

6.3.16.4 Retainer Plates: The use of retainer plates is a very low cost provision to improve the seismic response of elevators.

RELATED CONCERNS:

Maintenance: Mechanical and electrical devices installed to satisfy the requirements of the *Provisions* (e.g., resilient mounting *components* or certain protecting devices) require maintenance to ensure their reliability and provide the protection in case of a seismic event for which they are designed. Specifically, rubber-in-shear mounts or spring mounts (if exposed to weathering) may deteriorate with time and, thus, periodic testing is required to ensure that their damping action will be available during an earthquake. Pneumatic mounting devices and electric switchgear must be maintained free of dirt and corrosion. How a regulatory agency could administer such periodic inspections was not determined and, hence, requirements to cover this situation have not been included.

Tenant Improvements: It is intended that the requirements in Chapter 6 also apply to newly constructed tenant improvements that are listed in Tables 6.2.2 and 6.3.2 and that are installed at any time during the life of the structure.

Minimum Standards: Criteria represented in the *Provisions* represent minimum standards. They are designed to minimize hazard for occupants and to improve the likelihood of functioning of facilities required by the community to deal with the consequences of a disaster. They are not designed to protect the owner's investment, and the designer of the facility should review with the owner the possibility of exceeding these minimum standards so as to limit his economic risk. The risk is particularly acute in the case of sealed, air-conditioned structures where downtime after a disaster can be materially affected by the availability of parts and labor. The parts availability may be significantly worse than normal because of a sudden increase in demand. Skilled labor also may be in short demand since available labor forces may be diverted to high priority structures requiring repairs.

Architect-Engineer Design Integration: The subject of architect-engineer design integration is being raised because it is believed that all members of the profession should clearly understand that Chapter 6 is a compromise based on concerns for enforcement and the need to develop a simple, straightforward approach. It is imperative that from the outset architectural input concerning definition of occupancy classification and the required level of seismic resistance be properly integrated with the approach of the structural engineer to seismic safety if the design profession as a whole is to make any meaningful impact on the public conscience in this issue. Accordingly, considerable effort was spent in this area of concern. It is hoped that as the design profession gains more knowledge and sophistication in the use of seismic design, it will collectively be able to develop a more comprehensive approach to earthquake design requirements.

REFERENCES

ANCO Engineers, Inc. 1983. *Seismic Hazard Assessment of Non-Structural Components -- Phase I*, Final Report for National Science Foundation from ANCO Engineers, Inc., Culver City, California, September.

Bachman, R. E, and R. M. Drake. 1996. A Study To Empirically Validate the Component Response Modification Factors in the 1994 *NEHRP Provisions* Design Force Equations for Architectural, Mechanical, and Electrical Components, letter report to the National Center for Earthquake Engineering Research, July.

Bachman, R. E., R. M. Drake, and P. J. Richter. 1993. *1994 Update to 1991 NEHRP Provisions* *for Architectural, Mechanical, and Electrical Components and Systems*, letter report to the National Center for Earthquake Engineering Research, February 22.

Bouwkamp, J.G. and Meehan, J.F. 1960. "Drift Limitations Imposed by Glass;" Proceedings of the Second World Conference on Earthquake Engineering; Tokyo, Japan; pp. 1763-1778.

Drake, R. M., and R. E. Bachman. 1996. "*NEHRP Provisions* for 1994 for Nonstructural Components," *ASCE Journal of Architectural Engineering*, March.

Drake, R. M., and R. E. Bachman. 1995. "Interpretation of Instrumented Building Seismic Data and Implications for Building Codes," in *Proceedings of the 1995 SEAoC Annual Convention.*

Drake, R. M., and R. E. Bachman. 1994. "1994 *NEHRP Provisions* for Architectural Mechanical, and Electrical Components," in *Proceedings*, 5th United States National Conference on Earthquake Engineering.

Housner, G. W., and M. A. Haroun. 1980. "Seismic Design of Liquid Storage Tanks" in *ASCE Convention Proceedings.*

Wright, P.D. 1989. "The Development of a Procedure and Rig for Testing the Racking Resistance of Curtain Wall Glazing;" Building Research Association of New Zealand (BRANZ) Study Report No. 17; p. 18.

Chapter 7 Commentary

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL: The minimum foundation design requirements that might be suitable when any consideration must be given to earthquake resistance are set forth in Chapter 7. It is difficult to separate foundation requirements for minimal earthquake resistance from the requirements for resisting normal vertical loads. In order to have a minimum base from which to start, this chapter assumes compliance with all basic requirements necessary to provide support for vertical loads and lateral loads other than earthquake. These basic requirements include, but are not limited to, provisions for the extent of investigation needed to establish criteria for fills, slope stability, expansive soils, allowable soil pressures, footings for specialized construction, drainage, settlement control, and pile requirements and capacities. Certain detail requirements and the allowable stresses to be used are provided in other chapters of the *Provisions* as are the additional requirements to be used in more seismically active locations.

7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS: The resisting capacities of the foundations must meet the provisions of Chapter 7.

7.2.1 Structural Materials: The strength of foundation *components* subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements must be as determined in Chapters 8, 9, 10, 11, or 12.

7.2.2 Soil Capacities: This section requires that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil.

7.3 SEISMIC DESIGN CATEGORIES A AND B: There are no special seismic provisions for the design of foundations for buildings assigned to Categories A and B.

7.4 SEISMIC DESIGN CATEGORY C: Extra precautions are required for the seismic design of foundations for buildings assigned to Category C.

7.4.1 Investigation: Potential site hazards such as fault rupture, liquefaction, ground deformation, and slope instability should be investigated when the size and importance of the project so warrants. In this section, procedures for evaluating these hazards are reviewed.

Surface Fault Rupture: Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, generally occur along traces of previously active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these *components*. The following commentary summarizes procedures to follow or consider when assessing the hazard of surface fault rupture. This commentary is based in large part on Appendix C of California Division of Mines and Geology (CDMG) Special Publication 42, 1988 Revision (Hart, 1988).

Assessment of Surface Faulting Hazard: The evaluation of fault hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart, 1988): "An active fault has had displacement in Holocene time (last 11,000 years)."

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, and the amount and character of past displacements. Identification and characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies should include trenching to accurately locate, document, and date fault features.

Suggested Approach for Assessing Surface Faulting Hazard: The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. However, it is not expected that all of the following methods would be used in a single investigation:

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.

2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography, vegetation and soil contrasts, and other lineaments of possible fault origin. Predevelopment air photos are essential to the detection of fault features.

3. A field reconnaissance study generally is required which includes observation and mapping of geologic and soil units and structures, geomorphic features, springs, and deformation of man-made structures due to fault creep. This study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site.

4. Subsurface investigations usually are needed to evaluate fault features. These investigations include trenches, pits, or bore holes to permit detailed and direct observation of geologic units and fault features.

5. The geometry of fault structures may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require a knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone never prove the absence of a fault and they do not identify the recency of activity.

6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis (^{14}C, K-Ar), stratigraphic correlation (fossils, mineralology) soil profile development, paleomagnetism

(magnetostratigraphy), or other age-dating techniques to date the age of faulted or unfaulted units or surfaces.

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of hazardous faults on or near the site.

2. The type, amount, and sense of displacement of past surface faulting episodes should be documented including sense and magnitude of displacement, if possible.

3. From this documentation, estimates can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from empirical correlations between fault displacement and fault length or earthquake magnitude published by Bonilla et al. (1984) or by Slemmons et al. (1989). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected).

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimates reports, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, the general setback requirement in California is a minimum of 50 feet from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safety applied (*California Administrative Code*, Title 14, Sec. 3603A).

2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved. Where sufficient geologic data have been developed to accurately locate the zone containing active fault traces and the zone is not complex, a 50-foot setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may

contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

Mitigation of Surface Faulting Hazards: There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture. In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

Liquefaction: Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. For example, many structures in Niigata, Japan, suffered major damage as a consequence of liquefaction during the 1964 earthquake. Loss of bearing strength, differential settlement, and differential horizontal displacement due to lateral spread were the direct causes of damage. Many structures have been similarly damaged by differential ground displacements during U.S. earthquakes such as the San Fernando Valley Juvenile Hall during the 1971 San Fernando, California, earthquake and the Marine Sciences Laboratory at Moss Landing, California, during the 1989 Loma Prieta event. Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and mitigating the hazard by designing to resist ground displacement, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard.

Evaluation of Liquefaction Hazard: Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question.

The following possible methods for calculating the factor of safety against liquefaction have been proposed and used to various extents:

1. Analytical Methods -- These methods typically rely on laboratory test results to determine either liquefaction resistance or soil properties that can be used to predict the development of liquefaction. Various equivalent linear and nonlinear computer methods are used with the laboratory data to evaluate the potential for liquefaction. Because of the considerable difficulty in obtaining undisturbed samples of liquefiable sediment for laboratory evaluation of constitutive soil properties, the use of analytical methods, which rely on accurate constitutive properties, usually are limited to critical projects or to research.

2. Physical Modeling -- These methods typically involve the use of centrifuges or shaking tables to simulate seismic loading under well defined boundary conditions. Soil used in the model is reconstituted to represent different density and geometrical conditions. Because of

difficulties in precisely modeling in-situ conditions at liquefiable sites, physical models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well defined boundary conditions.

3. Empirical Procedures -- Because of the difficulties in analytically or physically modeling soil conditions at liquefiable sites, empirical methods have become a standard procedure for determining liquefaction susceptibility in engineering practice. Procedures for carrying out a liquefaction assessment using the empirical method are given by the National Research Council (1985).

For most empirical methods, the average earthquake-induced cyclic shear stress is estimated from a simple equation or from dynamic response analyses using computer programs such as SHAKE and DESRA. The induced cyclic shear stress is estimated from the peak horizontal acceleration expected at the site using the following simple equation:

$$\frac{\tau}{\sigma_o'} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma_o'}\right) r_d \qquad \text{(C7.4.1-1)}$$

where (a_{max}/g) = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity, σ_o = the vertical total stress in the soil at the depth in question, σ_o' = the vertical effective stress at the same depth, and r_d = deformation-related stress reduction factor.

The chart reproduced in Figure C7.4.1-1 is used to estimate r_d.

To determine liquefaction resistance of sandy soils, the induced cyclic stress ratio computed from Eq. C7.4.1-1 is compared to the cyclic stress ratio required to generate liquefaction in the soil in question for a given earthquake of magnitude M. The most common technique for estimating liquefaction resistance is from an empirical relationship between cyclic stress ratio required to cause liquefaction and normalized blow count, $(N_1)_{60}$.

The most commonly used empirical relationship, compiled by Seed et al. (1985), compares $(N_1)_{60}$ from sites where liquefaction did or did not develop during past earthquakes. Figure C7.4.1-2 shows the most recent (1988) version of this relationship for M = 7-1/2 earthquakes. On that figure, cyclic stress ratios calculated for various sites are plotted against $(N_1)_{60}$.

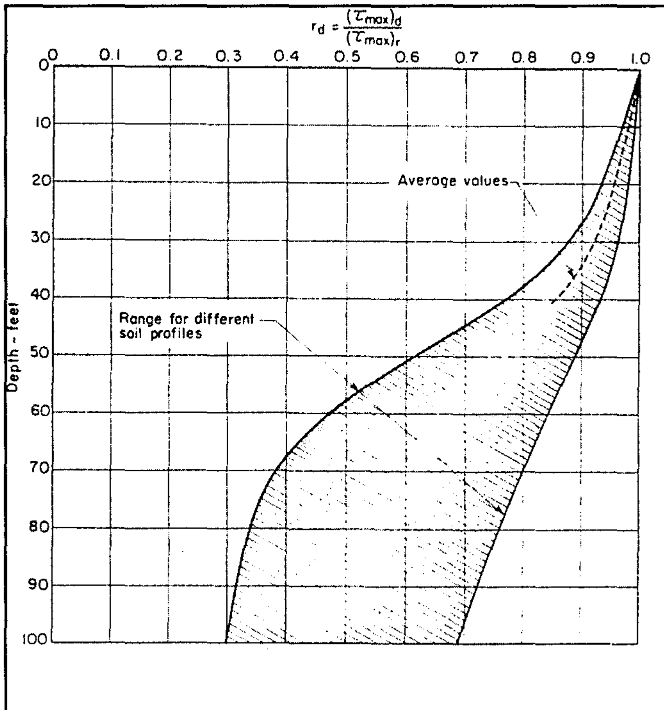


FIGURE C7.4.1-1 Range of values for r_d for different soil properties (after Seed and Idriss, 1971).

Solid dots represent sites where liquefaction occurred and open dots represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. Curves are given for sediments with various fines contents.

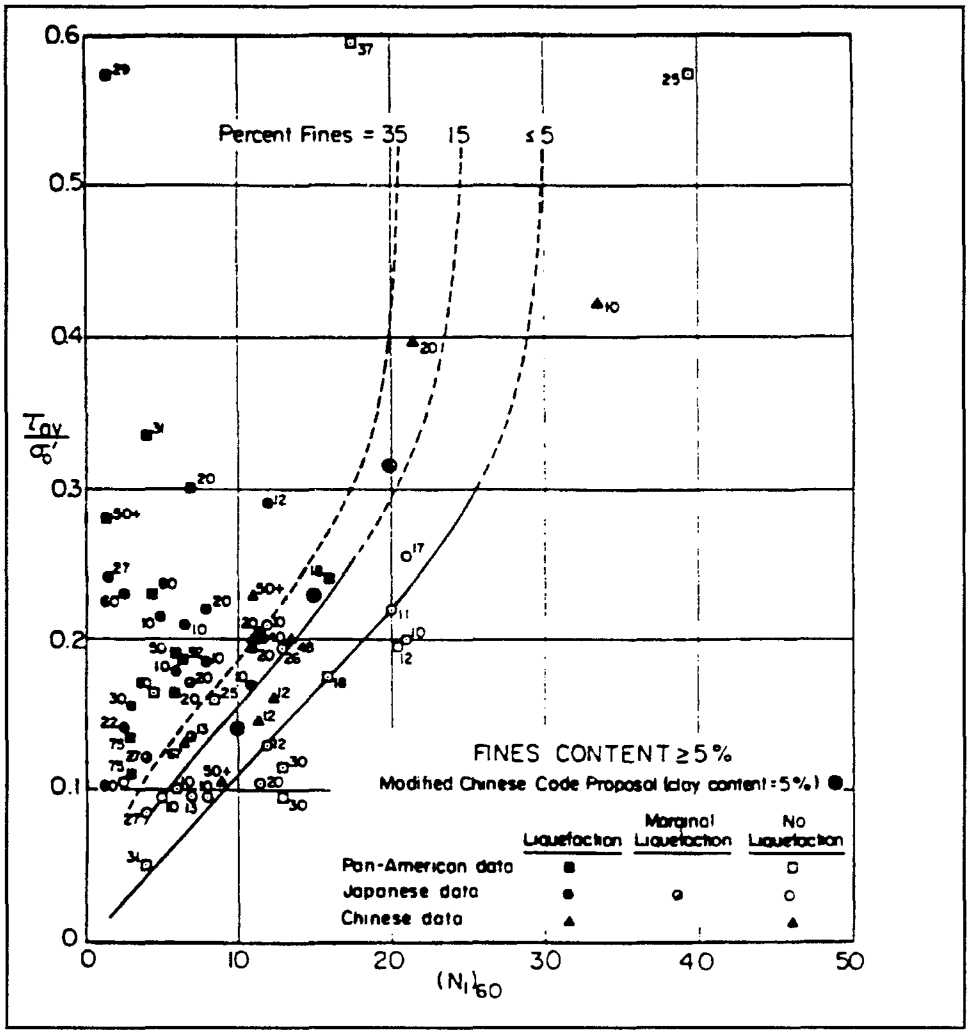


FIGURE C7.4.1-2 Relationship between stress ratios causing liquefaction and N_1 values for silty sands for M = 7-1/2 earthquakes.

Although the curves drawn by Seed et al. (1985) envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. Consequently, a factor of safety of 1.2 to 1.5 is appropriate in engineering design. The factor to be used is based on engineering judgment with appropriate consideration given to type and importance of structure and potential for ground deformation.

The maximum acceleration, a_{max}, commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the a_{max} used in Eq. C7.4.1-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop. Alternatives for obtaining a_{max} are: (1) from standard peak acceleration attenuation curves valid for comparable soil conditions; (2) from standard peak acceleration attenuation curves for rock, corrected for site amplification or deamplification by means of standard amplification curves or computerized site response analysis such as described in the "Chapter 1 Commentary"; (3) obtaining first the value of effective peak acceleration, A_a, for rock depending on the map area where the site is located and then multiplying this value by a factor between 1 and 3 as discussed in the "Chapter 1 Commentary" to determine a_{max}; (4) from probabilistic maps of a_{max} with or without correction for site amplification or deamplification depending on the rock or soil conditions used to generate the map.

The magnitude, M, needed to determine a magnitude scaling factor from Figure C7.4.1-3 should correspond to the size of the design or expected earthquake selected for the liquefaction evaluation. If Alternative 3 or 4 is selected, the definition of M is not obvious and additional studies and considerations are necessary. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety F_L in Eq. C7.4.1-3) is determined **jointly** by a_{max} and M. Because of the longer duration of strong ground-shaking, large distant earthquakes may generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

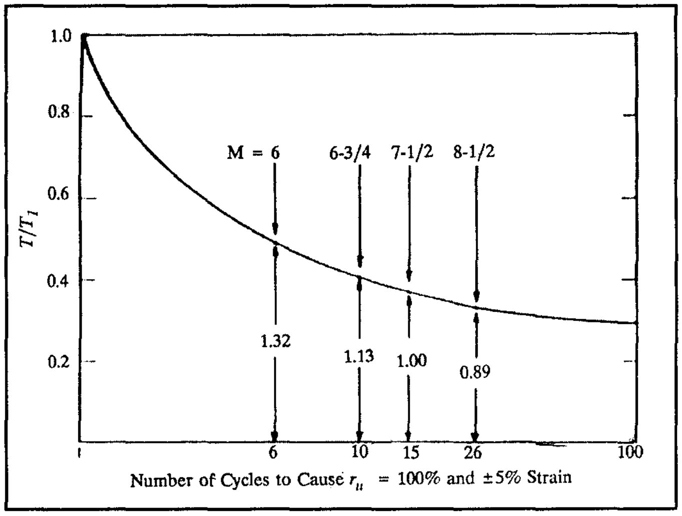


FIGURE C7.4.1-3 Representative relationship between T/T_1 and number of cycles required to cause liquefaction (after Seed et al., 1983).

The corrected blow count, $(N_1)_{60}$, required for evaluation of soil liquefaction resistance is commonly determined from measured standard penetration resistance, N_m, but may also be determined from cone penetration test (CPT) data using standard correlations to estimate N_m values from the CPT measurements. The corrected blow count is calculated from N_m as follows:

$$(N_1)_{60} = C_n \left(\frac{ER_m}{60} \right) N_m \tag{C7.4.1-2}$$

where C_n = a factor that corrects N_m to an effective overburden pressure of 1 tsf and ER_m = the rod energy ratio for the type of hammer and release mechanism used in the measurement of N_m.

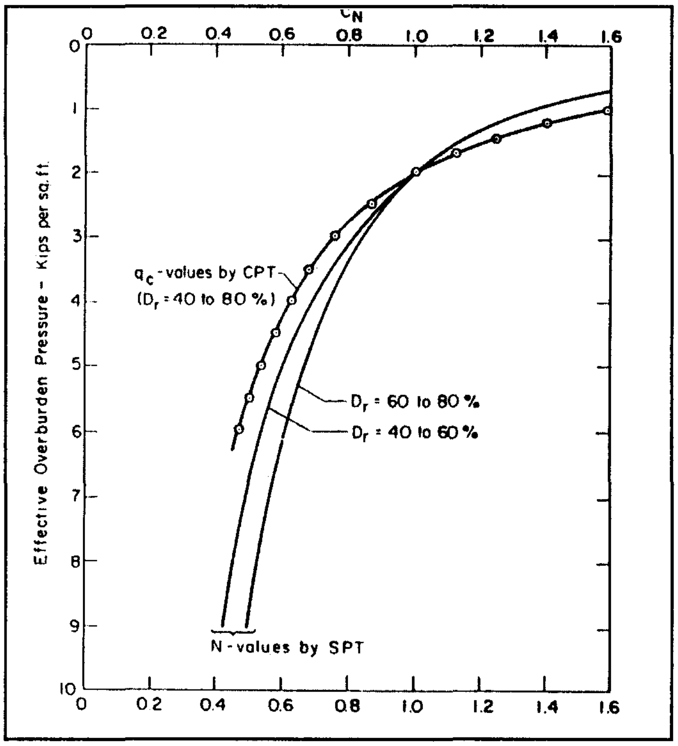


FIGURE C7.4.1-4 Chart for C_n (after Seed et al., 1985).

The curve plotted in Figure C7.4.1-4 is typically used to evaluate C_n. Measured hammer energies or estimates of hammer energies from tabulations such as those in Table C7.4.1 are used to define ER_m. An additional correction should be made to $(N_1)_{60}$ for shallow soil layers where the length of drilling rod is 10 feet or less. In those instances, $(N_1)_{60}$ should be reduced by multiplying by a factor of 0.75 to account for poor hammer-energy transfer in such short rod lengths.

Because a variety of equipment and procedures are used to conduct standard penetration tests in present practice and because the measured blow count, N_m, is sensitive to the equipment and procedures used, the following commentary and guidance with respect to this test is given. Special attention must be paid to the determination of normalized blow count, $(N_1)_{60}$, used in Figure C7.4.1-2. When developing the empirical relation between blow count and liquefaction resistance, Seed and his colleagues recognized that the blow count from SPT is greatly influenced by factors such as the method of drilling, the type of hammer, the sampler design, and the type of mechanism used for lifting and dropping the hammer. The magnitude of variations is shown by the data in Table C7.4.1.

TABLE C7.4.1 Summary of Rod Energy Ratios for Japanese SPT Procedures (after Seed et al., 1985)

Study	Mechanical Trip System (Tonbi)	Rope and Pulley
Nishizawa et al.	80-90	63-72
Decker, Holtz, and Kovacs	76	--
Kovacs and Salomone	80	67
Tokimatsu and Yoshimi	76[a]	--
Yoshimi and Tokimatsu, Yoshimi et al., Oh-Oka	--	--
Adopted for this study	78	67

[a] Equivalent rod energy ratio if rope and pulley method is assumed to have an energy ratio of 67 percent and values for mechanical trip method are different from this by a factor of 1.13.

In order to reduce variability in the measurement of N, Seed et al. (1983 and 1985) suggest the following procedures and specifications for the SPT test for liquefaction investigations:

1. The impact should be delivered by a rope and drum system with two turns of the rope around the rotating drum to lift a hammer weighing 140 lb or, more preferably, a drive system should be used for which ER_m has been measured or can be reliably estimated.
2. Use of a hole drilled with rotary equipment and filled with drilling mud. The hole should be approximately 4 in. in diameter and drilled with a tricone or baffled drag bit that produces upward deflection of the drilling fluid to prevent erosion of soil below the cutting edge of the bit.
3. In holes less than 50 feet deep, A or AW rod should be used; N or NW rod should be used in deeper holes.
4. The split spoon sampling tube should be equipped with liners or otherwise have a constant internal diameter of 1-3/8 inch.
5. Application of blows should be at a rate of 30 to 40 blows per minutes. (Some engineers suggest a slower rate of 20 to 30 blows per minute since it is easier to achieve and control and gives comparable results.) The blow count, N_m, is determined by counting the blows required to drive the penetrometer through the depth interval of 6 to 18 in. below the bottom of the hole.

Failure to follow these standard guidelines introduces large uncertainties into liquefaction estimates.

The curves in Figure C7.4.1-2 were developed from data for magnitude 7.5 earthquakes and are only valid for earthquakes of that magnitude. For larger or smaller earthquakes, the cyclic stress ratios determined from Figure C7.4.1-2 are corrected for magnitude by multiplying the determined cyclic stress ratio by a magnitude scaling factor taken from Figure C7.4.1-3. As the magnitude increases, the scaling factor decreases. For example, for an $(N_1)_{60}$ of 20, a clean sand (fines content < 5 percent) and an earthquake magnitude of 7.5, the CSRL determined from Figure C7.4.1-2 is 0.22. For the same site conditions but for a magnitude 8.0 earthquake, a CSRL of 0.20 is obtained after applying the magnitude scaling factor of 0.89 determined from Figure C7.4.1-3.

Soils composed of sands, silts, and gravels are most susceptible to liquefaction while clayey soils generally are immune to this phenomenon. The curves in Figure C7.4.1-2 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered. For soils containing more than 35 percent fines, the curve in Figure C7.4.1-2 for 35 percent fines should be used provided the following criteria developed by Seed et al. (1983) are met (i.e, the weight of soil particles finer than 0.005 mm is less than 15 percent of the dry weight of a specimen of the soil, the liquid limit of soil is less than 35 percent, and the moisture content of the in-place soil is greater than 0.9 times the liquid limit.

In summary, the procedure for evaluation of liquefaction resistance for a site is as follows: First, from a site investigation determine the measured standard penetration resistance, N_m, the percent fines, the percent clay (> 0.005 mm), the natural moisture content, and the liquid limit of the sediment in question. Check the measured parameters against the fines content and moisture criteria listed above to assure that the sediment is of a potentially liquefiable type. If so, correct N_m to $(N_1)_{60}$ using Eq. C7.4.1-2 and use Figure C7.4.1-2 to determine the cyclic stress ratio required to cause liquefaction for a magnitude 7.5 earthquake. Then correct that value using the appropriate magnitude scaling factor. That product is the cyclic stress ratio required to cause liquefaction in the field (CSRL). Next, calculate the cyclic stress ratio (CSRE) that would be generated by the expected earthquake using Eq. C7.4.1-1. Then compute the factor of safety, F_L, against liquefaction from the equation:

$$F_L = \frac{CSRL}{CSRE} \tag{C7.4.1-3}$$

If F_L is greater than one, then liquefaction should not develop. If at any depth in the sediment profile, F_L is equal to or less than one, then there is a liquefaction hazard. As noted above, a factor of safety of 1.2 to 1.5 is appropriate for building sites with the factor selected depending on the importance of the structure and the potential for ground displacement at the site.

Evaluation of Potential for Ground Displacements: Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support and/or ground deformation does this phenomenon become important to structural design. Loss of bearing capacity, flow failure, lateral spread, ground oscillation, and ground settlement are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described by the National Research Council (1985). The type of failure and amount of ground displacement are a function of several parameters including the thickness and extent of the liquefied layer, the thickness of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects (ground fissures and sand boils) for level sites. These criteria may be used for noncritical or nonessential structures on level sites. Additional analysis should be required for critical or essential structures.

Loss of Bearing Strength: Loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. General guidance for how deep or how thin the layers must be has not yet been developed. A geotechnical engineer, experienced in liquefaction hazard assessment, should be consulted to provide such guidance. Although loss of bearing strength may not be a hazard for deeply buried liquefiable layers, liquefaction-induced ground settlements or lateral-spread displacements could still cause damage and should be evaluated.

Ground Settlement: Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this commentary to outline that procedure which, although explicit, has several rather complex steps. For saturated or dry granular soils in a loose condition, their analysis suggests that the amount of ground settlement could approach 3 to 4 percent of the thickness of the loose soil layer. The Tokimatsu and Seed technique is recommended for estimating earthquake-induced ground settlement at sites underlain by granular soils and can be applied whether liquefaction does or does not occur.

Horizontal Ground Displacement: Only primitive analytical and empirical techniques have been developed to date to estimate ground displacement, and no single technique has been widely accepted or verified for engineering design. Analytical techniques generally apply Newmark's analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the residual strength of the deforming soil. Alternatively, nonlinear finite element methods have been used to predict deformations. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. The liquefaction severity index (LSI) correlation of Youd and Perkins (1987) provides a conservative upper bound for displacement for most natural soils (Figure C7.4.1-5; curves noted for various earthquakes are calculated from the equation on the figure). In this procedure, maximum horizontal displacement of lateral spreads in late Holocene fluvial deposits are correlated against earthquake magnitude and distance for the seismic source. The data are from the western United States and the correlation is valid only for that region. Because maximum displacements at very liquefiable sites were used in the LSI analysis, displacements predicted by that technique are conservative in that they predict an upper bound displacement for most natural deposits. Displacements may be greater, however, on uncompacted fill or extremely loose natural deposits.

The ground motions to be primarily considered in evaluating liquefaction potential are consistent with the *design earthquake* motions used in structural design. The structural design should be consistent with liquefaction-induced deformations resulting from those ground motions.

Liquefaction-induced deformations are not directly proportional to ground motions and may be more than 50 percent higher for maximum considered earthquake ground motions. The liquefaction potential and resulting deformations for ground motions consistent with the maximum considered earthquake should also be evaluated and, while not required in the *Provisions*, should be used by the registered design professional in checking for building damage that may result in collapse. In addition, Seismic Use Group III structures should be designed to retain a significant margin against collapse following liquefaction-induced deformations resulting from maximum considered earthquake ground motions.

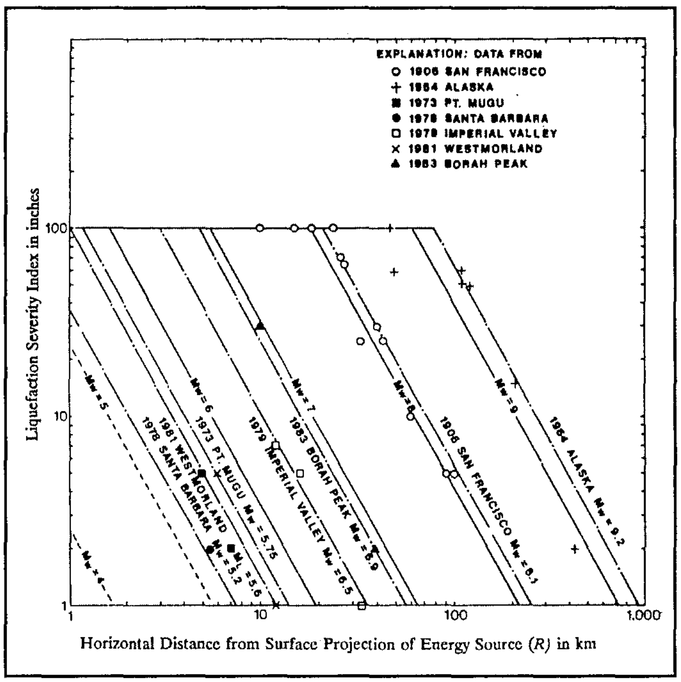


FIGURE C7.4.1-5 LSI from several western U.S. and Alaskan earthquakes plotted against horizontal distance from seismic energy sources (after Youd and Perkins, 1987).

The following further information is given for general guidance for ground conditions and range of displacements commonly associated with liquefaction-induced ground failures (National Research Council, 1995; Barlett and Youd, 1995):

1. Flow failures generally develop in loose saturated sands or silts on slopes greater than 3 degrees (5 percent) and may displace large masses of soil tens of meters. Standard limit equilibrium slope stability analyses may be used to assess flow failure potential with the residual strength used as the strength parameter in the analyses. The residual strength may be determined from empirical correlations such as that published by Seed and Harder (1989).

2. Lateral spreads generally develop on gentle slopes between 0.5 and 3 degrees (0.1 and 5 percent) and may induce up to several feet of lateral displacement. Empirical correlations have been developed by Bartlett and Youd (1995) to estimate lateral ground displacement due to liquefaction. Analytical procedures using appropriately reduced (residual) strengths of soils also are available to estimate displacements. These procedures range from simplified Newmark-type sliding block methods (e.g., Newmark, 1985; Makdisi and Seed, 1978) to more sophisticated finite element analyses. In general, the empirical correlations are simple to apply, do not require data beyond the commonly compiled engineering site investigations, and are usually adequate for routine engineering applications.

3. Ground oscillation occurs on nearly flat surfaces where the slope is too gentle to induce permanent horizontal displacement. During an earthquake, however, ground oscillation generates transient vertical or horizontal displacements that may range up to a few feet. For example, ground oscillation caused the rather chaotic pattern of ground displacements that offset pavements, thrust sidewalks over curbs, etc., in San Francisco's Marina District following the 1989 Loma Prieta earthquake.

Mitigation of Liquefaction Hazard: With respect to liquefaction hazard, three mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties as discussed in Sec. 7.4.3. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, may receive very little soil support through the liquefied layer and may be subjected to transient lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 foot although releveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a few inches.

Evaluations of structural performance following two recent Japanese earthquakes, 1993 Hokkaido Nansei-Oki ($M = 8.2$) and 1995 (Kobe) Hyogo-Ken Nanbu ($M = 7.2$), indicate that small *structures* on shallow foundations performed well in liquefaction areas. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to several tenths of a meter of lateral spread displacement. Many small *structures* (mostly houses, shops, schools, etc.) were structurally undamaged although a few tilted slightly. Foundations for these *structures* consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

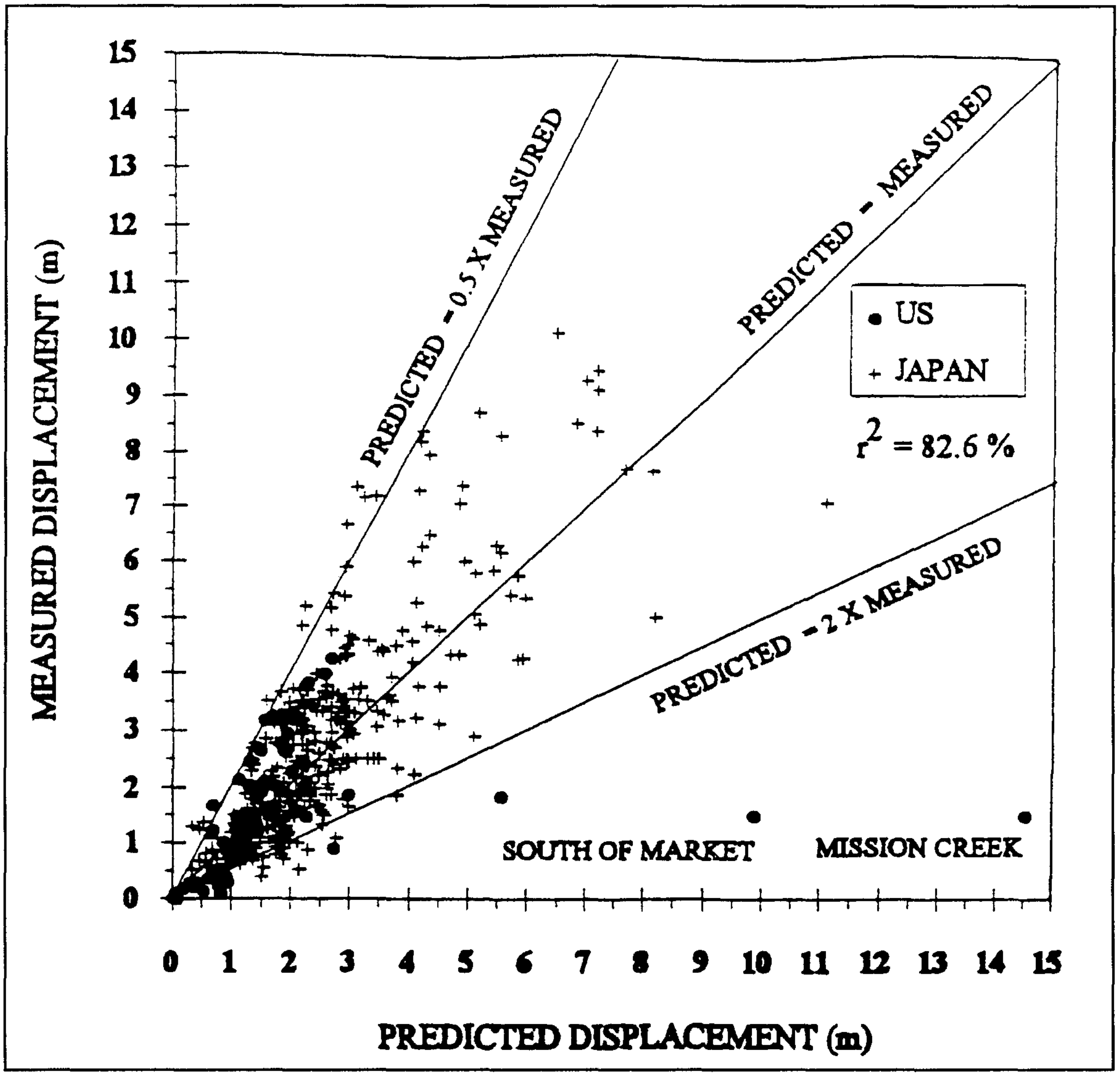


FIGURE C7.4.1-5 Measured displacements plotted against predicted displacements for U.S. and Japanese case-history data (after Bartlett and Youd, 1995).

Similarly, well reinforced foundations that would not fracture could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral (< 0.3 m) or vertical (< 0.05 m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural *components*.

Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

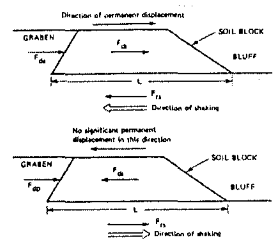
At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further explanation of these methods is given by the National Research Council (1985).

Slope Instability: The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using standard procedures.

For initial evaluation, the pseudostatic analysis may be used. (The deformational analysis described below, however, is now preferred.) In the pseudostatic analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the peak acceleration, a_{max}, or A_a. The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than one indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than one indicates that the slope will yield and slope deformation can be expected and a deformational analysis should be made using the techniques discussed below.

Deformational analyses yielding estimates of slope displacement are now accepted practice. The most common analysis uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration. The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. See Figure C7.4.1-6 for forces and equations used in analysis and Figure C7.4.1-7 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

The cumulative permanent displacement will depend on the yield acceleration as well as the intensity and duration of ground-shaking. As a general guide, a ratio of yield acceleration to maximum acceleration of 0.5 will result in slope displacements of the order of a few inches for typical magnitude 6.5 earthquakes and perhaps several feet of displacement for magnitude 8 earthquakes. Further guidance on slope displacement is given by Makdisi and Seed (1978).



F_{da} = drifing force due to active soil pressure

F_{di} = driving force due to earthquake inertia

F_{rs} = resisting force due to soil shear strength

F_{dp} = resisting force due to passive soil pressure

$$F_{di} = K_{\max} W$$

where K_{max} = maximum seismic coefficient and W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil and L = length of soil block

Yield seismic coefficient:

$$K_y = \frac{F_{rs} - F_{da}}{W}$$

FIGURE C7.4.1-6 Forces and equations used in analysis of translatory landslides for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985).

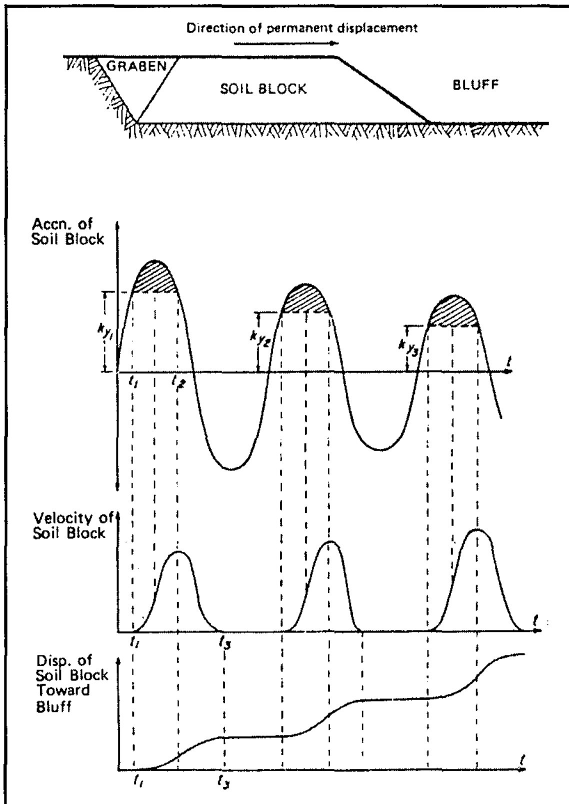


FIGURE C7.4.1-7 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966).

Mitigation of Slope Instability Hazard: With respect to slope instability, three general mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement. Techniques to stabilize a site include reducing the driving forces by grading and drainage of slopes and increasing the resisting forces by subsurface drainage, buttresses, ground anchors, or chemical treatment.

7.4.2 Pole-Type Structures: The use of pole-type structures is permitted. These structures are inherently sensitive to earthquake motions. Bending in the poles and the soil capacity for lateral resistance of the portion of the pole embedded in the ground should be considered and the design completed accordingly.

7.4.3 Foundation Ties: One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation that acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to $C_a/4$ times the larger pile cap or column load.

A common practice in some multistory buildings is to have major columns that run the full height of the building adjacent to smaller columns in the basement that support only the first floor slab. The coefficient applies to the heaviest column load.

Alternate methods of tying foundations together are permitted (e.g., using a properly reinforced floor slab that can take both tension and compression). Lateral soil pressure on pile caps is not a recommended method because the motion is imparted from soil to structure (not inversely as is commonly assumed), and if the soil is soft enough to require piles, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions.

If piles are to support structures in the air or over water (e.g., in a wharf or pier), batter piles may be required to provide stability or the piles may be required to provide bending capacity for lateral stability. It is up to the foundation engineer to determine the fluidity or viscosity of the soil and the point where lateral buckling support to the pile can be provided (i.e., the point where the flow of the soil around the piles may be negligible).

7.4.4 Special Pile Requirements: Special requirements for concrete or composite concrete and steel piles are given in this section. The piles must be connected to the pile caps with dowels.

Although unreinforced concrete piles are common used in certain areas of the country, their brittle nature when trying to conform to ground deformations makes their use in earthquake-resistant design undesirable. Nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie the elements together and to assist in load transfer at the top of pile to the pile cap. Experience has shown that concrete piles tend to hinge or shatter immediately below the pile cap so tie spacing is reduced in this area to better contain the concrete. In the case of the metal-cased pile, it is assumed that the metal casing provides containment and also a nominal amount of longitudinal reinforcement in the lower portion of the pile.

Bending stresses in piles caused by transfer of seismic motions from ground to structure need not be considered unless the foundation engineer determines that it is necessary. It has been a convenient analytical assumption to assume that earthquake forces originate in the building and are transmitted into and resisted by the ground. Actually the force or motion comes from the ground--not the structure. This makes the necessity of interconnecting footings more important, but what is desired is stability--not the introduction of forces.

Possibly the simplest illustration is shown in Figure C7.4.4. Consider a small structure subjected to an external force such as wind; the piles must resist that force in lateral pressure on the lee side of the piles. However, if the structure is forced to move during an earthquake, the wave motion is transmitted through the firmer soils, causing the looser soils at the surface and the building to move. For most structures, the structure weight is negligible in comparison to the weight of the

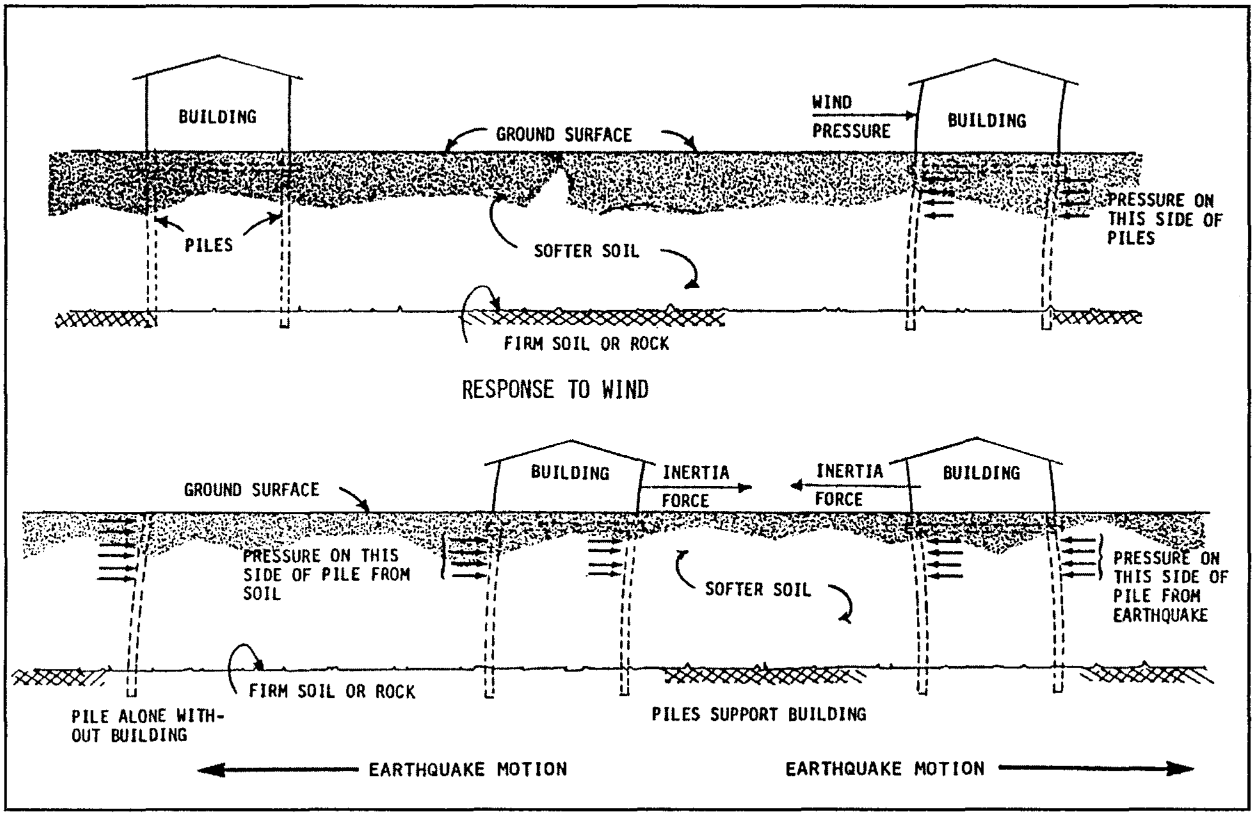


FIGURE C7.4.4 Response to earthquake.

surrounding surface soils. If an unloaded pile were placed in the soil, it would be forced to bend similar to a pile supporting a building.

The primary requirement is stability, and this is best provided by piles that can support their loads while still conforming to the ground motions and, hence, the need for ductility.

7.5 SEISMIC DESIGN CATEGORIES D , E, AND F: For Category D, E, or F construction, all the preceding provisions for Categories A, B, and C apply for the foundations, but the earthquake detailing is more severe and demanding. Adequate pile ductility is required and provision must be made for additional reinforcing to ensure, as a minimum, full ductility in the upper portion of the pile.

7.5.1 Investigation: In addition to the potential site hazard discussed in *Provisions* Sec. 7.4.1, consideration of lateral pressures on earth retaining structures shall be included in investigations for Seismic Design Categories D, E, and F.

Earth Retaining Structures: Increased lateral pressures on retaining structures during earthquakes have long been recognized; however, design procedures have not been prescribed in U.S. model building codes. Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. Damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that "engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior."

Seismic design analysis of retaining walls is discussed below for two categories of walls: "yielding" walls that can move sufficiently to develop minimum active earth pressures and "nonyielding" walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

Yielding Walls: At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Monobe-Okabe (M-O) seismic coefficient analysis (Monobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2(1 - k_v)K_{AE} \tag{7.5.1-1}$$

where: P_{AE} is the total (i.e., static + dynamic) lateral thrust, (γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \qquad (7.5.1\text{-}2)$$

or

$$K_{AE} = K_A + \Delta K_{AE} \qquad (7.5.1\text{-}2a)$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \qquad (7.5.1\text{-}2b)$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} \sim (3/4)K_h \qquad (7.5.1\text{-}3)$$

$$\Delta P_{AE} \sim (1/2)\gamma H^2(3/4)k_h \sim (3/8)k_h\gamma H^2 \qquad (7.5.1\text{-}3a)$$

where k_h is horizontal ground acceleration divided by gravitational acceleration. It is recommended that k_h be taken equal to the site peak ground acceleration that is consistent with design earthquake ground motions as defined in *Provisions* Sec. 7.5.3(i.e., $k_h = S_{DS}/2.5$). Equation 7.5.1-3 and 7.5.1-3a generally are referred to as the simplified M-O formulation.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and

Whitman (1970) recommended that the resultant dynamic thrust act at $0.6H$ above the base of the wall (i.e., inverted trapezoidal pressure distribution).

Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. There are a number of empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash and others (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards and others (1996).

Application of methods for evaluating tilting of yielding walls have been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgement. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation and displace only in the sliding mode.

Nonyielding Walls: Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (i.e., motions at less than half of the natural frequency of the unconstrained backfill), which would include many or most earthquake problems.

For uniform, constant k_h applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust, ΔP_E, acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \qquad (7.5.1\text{-}4)$$

The value of F is approximately equal to unity (e.g., Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \qquad (7.5.1\text{-}5)$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of $0.6H$ above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base. The subject of soil-wall interaction is addressed in the following sections. This section also provides further discussion on the applicability of the Wood and the M-O formulations.

Soil-Structure-Interaction Approach And Modeling for Wall Pressures: Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan and White (1998), among others, argue that the earth pressures acting on the walls of embedded structures during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (i.e., M-O method). Soil-structure interaction includes both a kinematic component--the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973)-- and an inertial component--the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et. al, 1975) and SASSI (Lysmer et al., 1981).

Ostadan and White (1998) have observed that for embedded structures subjected to ground shaking, the characteristics of the wall pressure amplitudes vs. frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by Ostadan and White (1998) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a "lower" and an "upper" bound, respectively.

Chang and others (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang and others indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure. Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (i.e., no significant rocking). In the latter case, the pressures given by the Wood (1973) formulation would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial *components* are out of phase.

Effect of Saturated Backfill on Wall Pressures: The previous discussions are limited to backfills that are not water-saturated. In current (1999) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (e.g., quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992).

A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present (1999), there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these "static" fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).

7.5.2 Foundation Ties: The additional requirement is made that spread footings on soft soil profiles should be interconnected by ties. The reasoning explained above under Sec. 7.4.3 also applies here.

7.5.4 Special Pile and Grade Beam Requirements: Additional pile reinforcing over that specified for Category C buildings is required. The reasoning explained above under Sec. 7.4.4 applies here.

Special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to pile design where portions of the foundation piles may be supported in soils such as loose granular materials and/or soft soils that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking.

For example:

1. Soil settlement at the pile-cap interface either from consolidation of soft soil prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.

2. Large deformations and/or reduction in strength resulting from liquefaction of loose granular materials can cause bending and/or conditions of free-standing columns.

3. Large deformations in soft soils can cause varying degrees of pile bending. The degree of pile bending will depend upon thickness and strength of the soft soil layer(s) and/or the properties of the soft/stiff soil interface(s).

Such conditions can produce shears and/or curvatures in piles that may exceed the bending capacity of conventionally designed piles and result in severe damage. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and these effects on concrete piles are further discussed by Shepard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a structure present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil although considerable judgment is necessary in utilizing this simple relationship in a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction with regard to pile foundation behavior.

The designer needs to consider the variation in soil conditions and driven pile lengths in providing for pile ductility at potential high curvature interfaces. Interaction between the geotechnical and structural engineers is essential.

It is prudent to design piles to remain functional during and following earthquakes in view of the fact that it is difficult to repair foundation damage. The desired foundation performance can be accomplished by proper selection and detailing of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves. Examples of designs of concrete piles include:

1.Use of a heavy spiral reinforcement and

2.Use of exterior steel liners to confine the concrete in the zones with large curvatures or shear stresses.

These provide proper confinement to ensure adequate ductility and maintenance of functionality of the confined core of the pile during and after the earthquake.

REFERENCES

American Society of Civil Engineers, Technical Committee on Lifeline Earthquake Engineering. 1998. *Seismic Guidelines for Ports*, edited by S. D. Werner, Monograph 12.

Bartlett, S. F., and T. L. Youd. 1992. *Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spread*, Technical Report NCEER-92-0021. Buffalo, New York: National Center for Earthquake Engineering Research.

Bartlett, S. F., and T. L. Youd. 1995. "Empirical Prediction of Liquefaction-Induced Lateral Spread." *Journal of Geotechnical Engineering ASCE* 121(4): 316-329.

Bertero, V. V, and H. A. Brauer, G. C. Fotinos, G. C. Gerwick, Jr., Y. T. Lin, and H. B. Seed. 1974. "Aseismic Design of Prestressed Concrete Piling." In *Proceedings of the 7th Congress of the Federation Internationale de la Precontrainte*, New York, May 25.

Bieganousky, W. A., and E. F. Marcuson, III. 1976. *Liquefaction Potential of Dams and Foundations: Report 1--Laboratory Standard Penetration Test on Reid Bedford Model and Ottawa Sands,* Report S-76-2. Waterways Experiment Station.

Bieganousky, W. A., and W. F. Marcuson, III. 1977. *Liquefaction Potential of Dams and Foundations: Report 2--Laboratory Standard Penetration Test on Platte River Sand and Standard Concrete Sand,* Report S-762. Waterways Experiment Station.

Bonilla, M. G., R. K. Mark, and J. J. Lienkaemper. 1984. "Statistical Relations Among Earthquake Magnitude, Surface Rupture Length, and Surface Fault Displacement." *Bulletin of the Seismological Society of America* 74(6):2379-2411.

Building Research Institute of Japan. *Report on the Damage by the 1978 Off Miyagi Prefecture Earthquake,* Report 86, pp. 319-341.

California Department of Transportation. 1971. *The San Fernando Earthquake--Field Investigation of Bridge Damage: Preliminary Report.* Sacramento: CALTRANS.

Castro, G. 1975. "Liquefaction and Cyclic Mobility of Saturated Sands." *Journal of the ASCE Geotechnical Engineering Division* 101 (GT6):551-569.

Chang, C. Y., M. S. Power, C. M. Mok, Y. K. Tang, and H. T. Tang. 1990. "Analysis of Dynamic Lateral Earth Pressures Recorded on Lotung Reactor Containment Model Structure," in *Proceedings, Fourth U.S. National Conference on Earthquake Engineering*, pp. 643-652. EERI.

Das, B. M. 1983. *Fundamentals of Soil Dynamics.* New York: Elsvier.

Dewa, Herose, Saito, and Ohira. 1980. "Earthquake Observations and Analysis of LNG Tank on Pile Foundations." In *Proceedings of the 7th World Conference on Earthquake Engineering,* Istanbul.

Earthquake Engineering Research Institute. 1986. "Geotechnical Engineering." In *Reducing Earthquake Hazards: Lessons Learned from Earthquakes*, Publication 86-02, pp. 23-52. El Cerrito, California: Earthquake Engineering Research Institute.

Ebeling, R. M., and E. E. Morrison. 1992. *The Seismic Design of Waterfront Retaining Structures*, Technical Report ITL-92-11. Vicksburg, Mississippi: Corps of Engineers Waterways Experiment Station.

Gerwick, B. C. Jr. 1982. "Seismic Design of Prestressed Concrete Piles." In *Proceedings of the 9th Congress of the Federation Internationale de la Preconstrainte*, Vol. 2, pp. 60-69.

Gerwick, B. C., Jr., and H. A. Brauner. 1970. *Design of High-Performance Prestressed Concrete Piles for Dynamic Loading.* New York: American Society for Testing and Materials.

Hardman, Scott L., and T. Leslie Youd. 1987. *State of the Art for Assessing Earthquake Hazards in the United States,* Miscellaneous Paper S-73-1. Vicksburg, Mississippi: U.S. Army Waterways Experiment Station.

Hart, E. W. 1988. *Fault-Rupture Hazard Zones in California*, Special Publication 42. Sacramento: California Division of Mines and Geology.

Idriss, I. M. 1989. "Response of Soft Soil Sites During Earthquakes." In *Proceedings of the H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 273-289.

Idriss, I. M. 1985 "Evaluating Seismic Risk in Engineering Practice." In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, Vol 1, p. 155.

Ishihara, Kenji. 1985. "Stability of Natural Deposits During Earthquakes." In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, Vol. 1, pp. 321-376.

Kishida, Hanazuto, and Nakai. 1980. "Damage of Reinforced Precast Concrete Piles During the Miyagi-Oki Earthquake of June 12, 1981." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul*, Vol. 9, p. 461.

Kishida. "Damage of Reinforced Concrete Buildings in Niigata City with Special Reference to Foundation Engineering." *Soils and Foundations* 6(1):71-86.

Koester, J. P., and A. G. Franklin. 1985. *Current Methodologies for Assessing the Potential for Earthquake Induced Liquefaction in Soils,* Report NUREG/CR-4430 RA. Washington, D.C.: U.S. Nuclear Regulatory Commission.

Kramer, S. L. 1996. *Geotechnical Earthquake Engineering*. New Jersey: Prentice Hall.

Lam, I., and G. R. Martin. 1986. *Seismic Design of Highway Bridge Foundations -- Design Procedures and Guidelines*, Report FHWA/RD-86/102. Washington, D.C.: FHWA.

Ledbetter, R. H. 1985. *Improvement of Liquefiable Foundation Conditions Beneath Existing Structures,* Technical Report REMR-GT-2. Vicksburg, Mississippi: U.S. Army Waterways Experiment Station.

Lysmer, J., M. Tabatabai-Rassi, F. Jajirian, S. Vahdani, and F. Ostadan. 1981. *SASSI — A System for Analysis of Soil-Structure Interaction*, Report UCB/GT/81-02. Berkeley: University of California Department of Engineering.

Lysmer, J., T. Udka, C. P. Tsai, and H. B. Seed. 1975. *FLUSH: A Computer Program for Approximate 2-D Analysis of Soil-Structure Interaction Problems*, Report EERC 75-30. Berkeley: University of California, Earthquake Engineering Research Center.

Makdisi, F. T., and H. B. Seed. 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering* ASCE 104(GT7):849-867.

Margason, E., and M. Holloway. 1977. "Pile Bending During Earthquakes." In *Proceedings of the 6th World Conference on Earthquake Engineering, New Dehli.*

Monobe, N. and H. Matsuo. 1929. "On the Determination of Earth Pressures During Earthquakes." In *Proceedings, World Engineering Congress.*

Nadim, F. 1980. "Tilting and Sliding of Gravity Retaining Walls," MS thesis, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge.

Nadim, F., and R. V. Whitman. 1984. "Coupled Sliding and Tilting of Gravity Retaining Walls During Earthquakes." In *Proceedings of the Eighth World Conference on Earthquake Engineering*, pp. 477-484.

National Research Council, Committee on Earthquake Engineering. 1985. *Liquefaction of Soils During Earthquakes*. Washington, D.C.: National Academy Press.

Nodar, Setsuo, and Satoshi Hayashi. 1980. "Damage to Port Structures in the 1978 Miyagi-Kenoki Earthquake." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul*, Vol. 9, p. 415.

Norris, G. M. 1994. "Seismic Bridge Pile Foundation Behavior," in *Proceedings, International Conference on Design and Construction of Deep Foundations*, FHWA, Vol. 1.

Okabe, S. 1926. "General Theory of Earth Pressure," *Journal of the Japan Society of Civil Engineers* 12,1.

Ostadan, F., and W. H. White. 1998. Lateral Seismic Soil Pressure — An Updated Approach. In Preproceedings of UJNR Workshop on Soil-Structures Interaction, U.S. Geological Survey, Menlo Park, California.

Park, R., and T. J. Falconer. 1983. "Ductility of Prestressed Concrete Piles Subjected to Seismic Loading." *PCI Journal* (September/October).

Poulos, S. J., G. Castro, and J. W. France. 1985. "Liquefaction Evaluation Procedures." *ASCE Journal of Geotechnical Engineering* 111(6).

Prakash, S. 1981. *Soil Dynamics*. New York: McGraw-Hill Book Co.

Prakash, S., Y. Wu, and E. A. Rafnsson. 1995. *Displacement Based Aseismic Design Charts for Rigid Walls*. Rolla, Missouri: Shamsher Prakash Foundation.

Richards, R. J., and D. Elms. 1979. "Seismic Behavior of Gravity Retaining Walls." *Journal of the Geotechnical Engineering Divisions, ASCE* 105(GT4): 449-464.

Richards, R., K. L. Fishman, and R. Devito. 1996. "Threshold Accelerations for Rotation or Sliding of Bridge Abutments," *Journal of Geotechnical Engineering*, Asce, 105, GT4, 449-464.

Rollins, Kyle M. 1987. The Influence of Buildings on Potential Liquefaction Damage. Ph.D. dissertation, University of California, Berkeley.

Seed, H. B. 1987. "Design Problems in Soil Liquefaction." *ASCE Journal of Geotechnical Engineering* 113(8).

Seed, H. B., I. Arango, and C. K. Chan. 1975. *Evaluation of Soil Liquefaction Potential During Earthquakes,* Report EERC 75-28. Berkeley, California: Earthquake Engineering Research Center.

Seed, H. B., and W. H. Peacock. 1971. "Test Procedures for Measuring Soil Liquefaction Characteristics." *Journal of the ASCE Soil Mechanics and Foundation Division* 97(SM8), Paper 8330.

Seed, H. B., and I. M. Idriss. 1971. "Simplified Procedure for Evaluating Soil Liquefaction Potential." *Journal of the ASCE Soil Mechanics and Foundations Division* 97(SM9):1249-1273.

Seed, H. B., Mori, Kenji, and C. K. Chan. 1977. "Influence of Seismic History on the Liquefaction of Sands." *Journal of the ASCE Geotechnical Engineering Division* (April).

Seed, H. B. 1976. "Evaluation of Soil Liquefaction Effects of Level Ground During Earthquakes." In *Liquefaction Problems in Geotechnical Engineering, Proceedings of the ASCE National Convention.*

Seed, H. B., K. Tokimatsu, L. F. Harder, and R. M. Chung. 1985. *The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations*, Report UBC/EERC-84/15. Berkeley, California: Earthquake Engineering Research Center.

Seed, H. B., I. M. Idriss, and I. Arango. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data." *ASCE Journal of Geotechnical Engineering* 109(3):458-482.

Seed, H. B., and I. M. Idriss. 1982. *Ground Motions and Soil Liquefaction During Earthquakes*, Vol. 5 of *Engineering Monographs on Earthquake Criteria, Structural Design, and Strong Motion Records*. El Cerrito, California: Earthquake Engineering Research Institute.

Seed, R. B., and L. F. Harder. 1989. "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength." In *Proceedings of the H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 351-376.

Seed, H. B., and R. V. Whitman. 1970. "Design of Earth Retaining Structures for Dynamic Loads," in *Proceedings, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures*, pp. 103-147. Ithaca, New York: Cornell University.

Shepard, D. A. 1983. "Seismic Design of Concrete Piling." *PCI Journal* (March/April).

Siddarthan, R., S. Ara, and G. Norris. 1992. "Simple Rigid Plastic Model for Seismic Tilting of Rigid Walls," *Journal of Structural Engineering*, ASCE, 469-487.

Slemmons, D. B., P. Bodin, and X. Zhang. 1989. "Determination of Earthquake Size from Surface Faulting Events." In *Proceedings of the International Seminar on Seismic Zonation*, Guangahou, China, pp. 157-169.

Soydemir, C., and M. Celebi. 1992. "Seismic Design of Buildings with Multi-Level Basements," in *Proceedings*, Tenth World Conference on Earthquake Engineering, Madrid Spain, Vol. 6, pp. 1731-1734.

Steedman, R. S., and X. Zeng. 1996. "Rotation of Large Gravity Retaining Walls on Rigid Foundations Under Seismic Loading." In *Analysis and Design of Retaining Walls Against Earthquakes*, ASCE Geotechnical Special Publication 60, edited by S. Prakash, pp. 38-56. New York: ASCE.

Sugimura. 1981. "Earthquake Damage and Design Method of Piles." In *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm.*

Sugimura. 1980. "Participation Factor of Horizontal Force Applied to Pile Foundation." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul*, Vol. 3, p. 443.

Tamura et al. 1973. "Damage of Bridges During the Tokachi-Oki Earthquake (1968)." In *Proceedings of the Japan Earthquake Engineering Symposium*, pp. 147-153.

Tokimatsu, K., and H. B. Seed. 1987. "Evaluation of Settlements in Sands Due to Earthquake Shaking." *Journal of Geotechnical Engineering* ASCE 113(8):861-878.

Veletsos, A., and A. H. Younan. 1994a. "Dynamic Soil Pressure on Rigid Vertical Walls," *Earthquake Engineering and Soil Dynamics*, 23:275-301.

Veletsos, A. and Younan, A. H. 1994b. "Dynamic Modeling and Response of Soil-Wall Systems," *Journal of Geotechnical Engineering*, ASCE, 120:12.

Whitman, R. V. 1991. "Seismic Design of Earth Retaining Structures," in *Proceedings*, Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.

Whitman, R. V., and S. Liao. 1985. *Seismic Design of Retaining Walls*, Miscellaneous Paper GL-85-1. Vicksburg, Mississippi: Corps of Engineers Waterways Experiment Station.

Wood, J. H. 1973. *Earthquake-Induced Soil Pressures on Structures*, Report EERL 73-05. Pasadena: California Institute of Technology.

Youd, T. L. 1993. *Liquefaction-Induced Lateral Spread Displacement.* NCEL Technical Note N-1862. Washington, D.C.: U.S. Navy.

Youd, T. L. 1989. "Ground Failure Damage to Buildings During Earthquakes." In *Foundation Engineering--Current Principles and Practices*, Vol. 1, pp. 758-770. New York: American Society of Civil Engineers.

Chapter 8 Commentary

STEEL STRUCTURE DESIGN REQUIREMENTS

8.1 REFERENCE DOCUMENTS: The reference documents presented in this section are the current specifications for the design of steel members, systems, and *components* in *buildings* as approved by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), the American Society of Civil Engineers (ASCE) and the Steel Joist Institute (SJI).

Revise the AISC Seismic Commentary Sec. C9.3 as follows: At the end of the second paragraph add the following: "This provision requires that the panel zone be proportioned using the method used to proportion the panel zone thickness of successfully tested connections. This should not be constructed to mean that the thickness is required to be the same as the tested connection, only that the same method must be used to proportion it. For example, if the test were preformed on a one-sided connection and the same beam and column sizes were used in two-sided connection, the panel zone would be twice as thick as that of the tested connection."

8.2 SEISMIC REQUIREMENTS FOR STEEL STRUCTURES:

8.3 SEISMIC DESIGN CATEGORIES A, B, AND C: Structures assigned to Seismic Design Categories A, B, and C do not require the same level of ductility capacity to provide the required performance as those assigned to the higher categories. For this reason, such structures are permitted to be designed using the requirements of any of the listed references, provided that the lower R value specified in Table 5.2.2 is used. Should the registered deisgn professional choose to use the higher R values in the table, it is required that the detailing requiorements for the higher Seismic Design Categories be used.

8.4 SEISMIC DESIGN CATEGORIES D, E, AND F: Structures assigned to these categories must be designed in anticipation of significant ductility demands that may be placed on the structures during their useful life. Therefore, structures in these categories are required to be designed to meet special detailing requirements as referenced in this section.

8.5 COLD-FORMED STEEL SEISMIC REQUIREMENTS: The allowable stress and allowable load levels in AISI are incompatible with the force levels in Chapter 5 of the *Provisions*. It is therefore necessary to modify the provisions of AISI for use with the *Provisions*. ANSI/ASCE 8-90 and SJI are both based on LRFD and thus are consistent with the force levels in Chapter 5 of the *Provisions*. As such, only minor modifications are needed to correlate those load factors for seismic loads to be consistent with the *Provisions*. The modifications of all of the reference documents affect only designs involving seismic loads.

8.6 LIGHT-FRAMED WALLS: The provisions of this section apply to buildings framed with cold-formed steel studs and joists. Lateral resistance is typically provided by diagonal braced (braced frames) or wall sheathing material. This section is only required for use in Seismic Design Categories D, E, and F. The required strength of connections is intended to assure that inelastic behavior will occur in the connected members prior to connection failure. Since pull-out of screws is a sudden or brittle type of failure, designs using pull-out to resist seismic loads are not permitted. Where diagonal members are used to resist lateral forces, the resulting uplift forces must be resolved into the foundation or other frame members without relying on the bending resistance of the track web. This often is accomplished by directly attaching the end stud(s) to the foundation, frame, or other anchorage device.

Table 8.6 presents nominal shear values for plywood and oriented strand board attached to steel stud wall assemblies. Design values are determined by multiplying the nominal values by a phi (ϕ) factor as presented in Sec. 8.6.5. These nominal values are based upon tests performed at Santa Clara University (Serrette, 1996). The test program included both cyclic and static tests; however, the values presented in Table 8.6 are based upon the cyclic tests as they are intended for use in seismic resistance. In low seismic areas where wind loads dominate, nominal values have been recommended for wind resistance by AISI based upon monotonic tests (AISI, 1996). The cyclic tests were performed using the assemblies that were determined to be the most critical from the static tests. The assemblies cyclically tested consisted of 3.5 x 1.625 inch C studs fabricated with ASTM A446 Grade A (33 ksi) with a minimum base metal thickness of 0.033 inch. Since the tests were conducted, ASTM A446 Grade A has been redesignated ASTM A653 SQ Grade 33. The test panels were four ft wide and 8 ft high, the sheathing material was applied vertically to only a single side of the studs, and there was no sheathing or bracing applied to the other side.

The cyclic tests were performed using a sequential phase displacement protocol under development at the time of the test by an *ad hoc* Committee of the Structural Engineers Association of Southern California. Nominal values were conservatively established by taking the lowest load in the last set of stable hysteretic loops. It is expected that subsequent testing of steel stud shear wall assemblies will reduce or modify some of the restrictive limits currently proposed for the use of the system such as the nominal maximum thickness of the studs of 0.043 inch, the aspect ratio of 2:1, and the ability to use sheathing on both sides of the wall.

8.7 SEISMIC REQUIREMENTS FOR STEEL DECK DIAPHRAGMS: Since the design values for steel deck are based on allowable loads, it is necessary to present a method of deriving design strengths. Two ϕ values are presented — 0.60 for steel deck that is mechanically attached and 0.50 for welded steel deck. These factors are consistent with current proposals being circulated for inclusion in updates of ANSI/ASCE 8-90.

8.8 STEEL CABLES: The provisions of Sec. 8.5 are virtually unchanged from previous editions. Although the provisions in ASCE 19 are dated, they are the only ones available and there was no sentiment to eliminate them from the *Provisions*. The allowable stress levels of steel cable structures specified in ASCE 19 are modified for seismic load effects. The value of $1.5T_4$ was chosen as a reasonable value to compare with increases given to other working stress levels.

REFERENCES:

Serette. 1996. *Shear Wall Values for Light Weight Steel Framing.* American Iron and Steel Institute.

Chapter 9 Commentary

CONCRETE STRUCTURE DESIGN REQUIREMENTS

9.1 REFERENCE DOCUMENT: The main concern of Chapter 9 is the proper detailing of reinforced concrete construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in ACI 318. The commentary for ACI 318 contains a valuable discussion of the rationale behind detailing requirements that is not repeated here.

9.1.1 Modifications to ACI 318: The modifications noted for ACI 318 are: changes in load factors necessary to coordinate with the equivalent yield basis of this document; additional definitions and provisions necessary for seismic design requirements for structural systems composed of precast elements; and changes that incorporate certain features of the detailing requirements for reinforced concrete that have been adopted into the 1997 *Uniform Building Code* and the 2000 *International Building Code.*

Included as Sec. 9.1.1.4 are two statements on reinforced concrete structural systems incorporating precast concrete elements. One statement refers to Sec. 9.1.1.12 where a new Sec.21.11 is inserted in ACI 318 to cover the design requirements for precast concrete special moment frames and special structural walls. The second statement is based on requirements from 1997 Uniform Building Code and provides design requirements for structures having precast concrete gravity load carrying systems .

For precast concrete special moment frames and special structural walls two design alternatives are permitted. One design alternative is emulation of monolithic reinforced concrete construction. The other alternative is the use of the unique properties of precast elements interconnected predominately by dry joints. For the first alternative Sec. 9.1.1.12 defines in provisions 21.11.2, 21.11.3 and 21.11.5 design procedures ensuring that the resulting structural systems have strength and stiffness characteristics equivalent to those for monolithic reinforced concrete construction. The existing code requirements for monolithic construction then apply for all but the connections. The second alternative, use of the unique properties of precast elements interconnected predominately by dry joints, was covered in an Appendix to Chapter 9 in the 1997 Provisions. Recent advances in the understanding of the seismic behavior of precast/prestressed concrete frame and wall structures, resulting from NIST (Cheok et al.1991,1997,1998), US-PRESSS (Priestley et al., 1991,1996,1999, Nakaki et al.,1999) and JAPAN-PRESSS research programs and the codification of acceptance testing procedures for verification of acceptable behavior by ITG-1 of ACI, 1999, have made possible the elimination of the penalties on the use of precast/prestressed concrete construction that were contained in the Appendix to the 1997 Provisions and the inclusion in Sec. 9.1.1.12 of a new provision 21.11.4 containing appropriate requirements for precast/prestressed concrete seismic-force-resisting systems based entirely on amendments to ACI 318.

Procedures for design of a seismic-force-resisting structural system composed of precast elements interconnected predominately by dry joints require prior acceptance testing of modules of the generic structural system because with the existing state-of-knowledge, it is inappropriate to propose code provisions without such verification. The complexity of structural systems, configurations and details possible with precast concrete elements requires:

1. Selecting functional and compatible details for connections and members that are reliable and can be built with acceptable tolerances;
2. Verifying experimentally the inelastic force-deformation relationships for welded, bolted, or grouted connections proposed for the seismic resisting elements of the building; and
3. Analyzing the building using those connection relationships and the inelastic reversed cyclic loading effects imposed by the anticipated earthquake ground motions.

Research conducted to date (Cheok and Lew, 1991; Elliott et al, 1992; Englekirk, 1987; French et al, 1989; BSSC, 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al, 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al, 1986; Stanton et al, 1991) documents concepts for design using dry connections and the behavior of structural systems and subassemblages composed of precast elements both at and beyond peak strength levels for non-linear reversed cyclic loadings, and provides the basis for the provisions for interconnected element design in Sec. 21.11.2, and Sec. 21.11.4 of Sec. 9.1.1.12.

Emulation of Monolithic Construction Using Strong Connections: For emulation of the behavior of monolithic reinforced concrete construction, Sec. 9.1.1.12 provides two alternatives. Sec. 21.1.3 in Sec. 9.1.1.12 covers structural systems with either "wet" or dry connections. Sec. 21.11.3.2 and 21.11.5 cover structural systems with "strong" connections.

For frame systems that use strong connections, Sec. 21.11.3.2 and 21.11.5 , the different connection categories envisaged are shown in Figure C9.1.1-1. Considerable freedom is given to locating the nonlinear action zones (plastic hinges), along the length of the precast member. Those hinges must be considered to have a length not less than half the member depth and must be separated from the connection by a distance of at least three quarters of the member depth. Wet-joint connections are permitted at the strong connection but not at the hinge location.

Provision 21.11.5.1 makes the strength required for a strong connection dependent on the distances hinges are separated from that connection, the strengths of those hinges and the nonlinear deformation mechanism envisaged. The conditions described by 21.11.5.1 for a beam to continuous column connection are shown in Figure C9.1.1- 2, which is an adaption of Figure R 21.3.4 of ACI 318. Because the strong connection must not yield or slip; its nominal strengths, S_n, in both flexure and shear must be greater than those corresponding to development of the probable strengths M_{pr1} and M_{pr2} at the hinge locations. Figure C9.1.1-2b, illustrates the situation for flexure. Per ACI 318 moments M_{pr1} and M_{pr2} are determined using a strength reduction factor of 1.0 and reinforcing steel stresses of at least $1.25fy$.

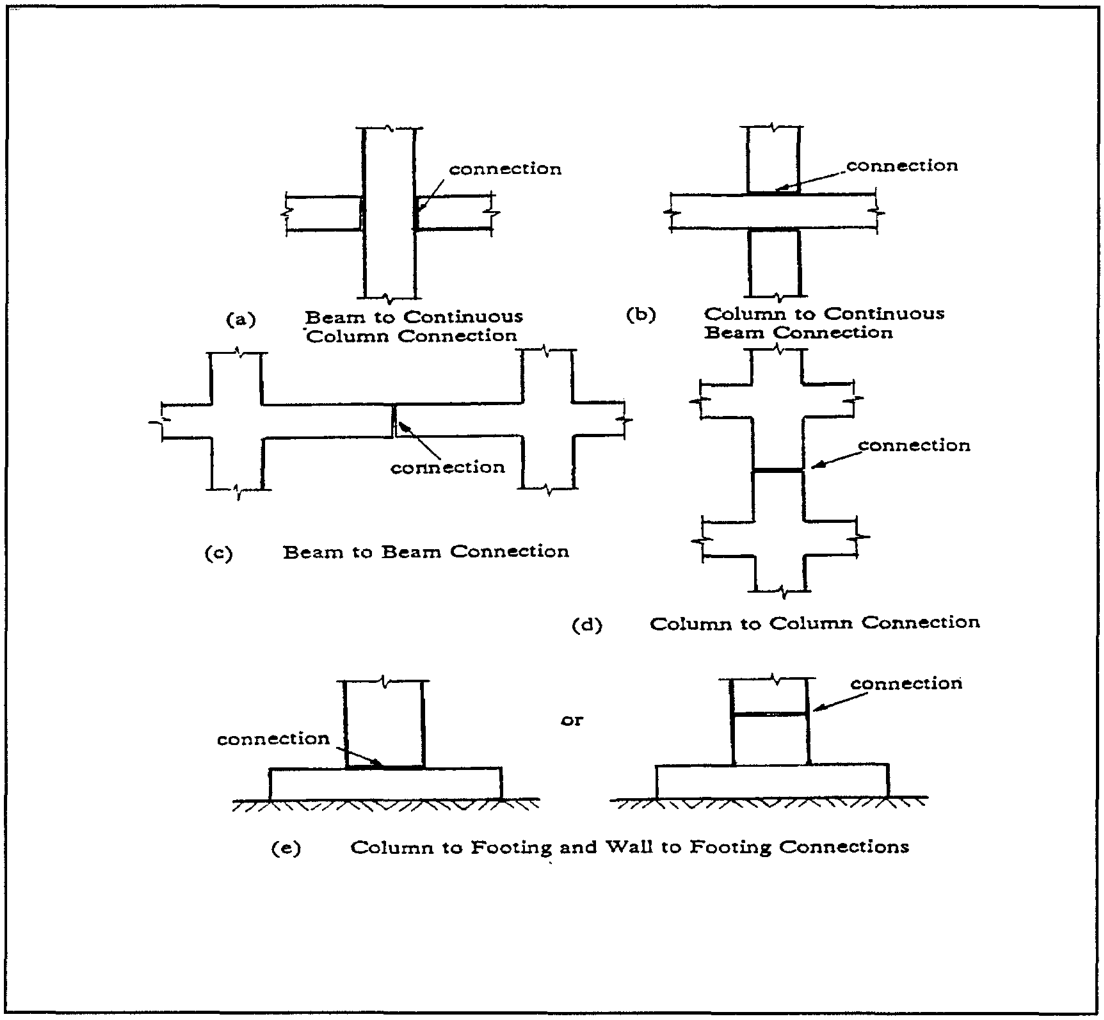


FIGURE C9.1.1-1 Connection categories.

For columns above the ground floor, moments at a joint may be limited by flexural strengths of the beams framing into that joint. However, for a strong column-weak beam deformation mechanism, dynamic inelastic analysis and studies of strong motion measurements have shown that beam end moments are not equally divided between top and bottom columns even where the columns have equal stiffness. Elastic analysis predicts moments as shown in Figure C9.1. 1-3b. Accordingly, provision 21.11.5.4 is included for the mid-height column connection. Further background information on the *Provisions* is provided in Ghosh et al.,1997.

Emulation of Monolithic Construction Using Ductile Connections: In Sec. 9.1.1.12 provision 21.11.3.1 covers the situation for both *frame* and panel systems where the connections used have adequate nonlinear response characteristics and it is not necessary to ensure plastic hinges remote form the connections. Usually physical testing is required to prove that a connection has the necessary nonlinear response characteristics. Warnes (1992) and Yee (1991) have documented one connection type that has such characteristics.

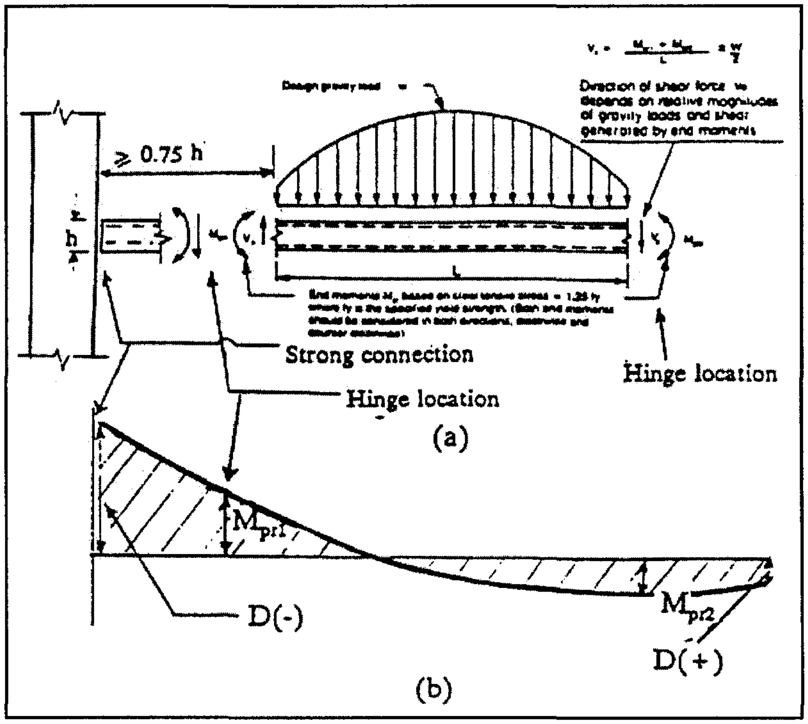


FIGURE C9.1.1-2 Design forces for strong connections between beams and continuous columns.

The designer needs to consider the likely deformations of any proposed precast structure vis-a-vis those of the same structure composed of monolithic reinforced concrete before claiming that the precast form emulates monolithic construction. For example, the designer might propose a shear wall that is composed of multiple precast panels over its length and height that are connected vertically but not horizontally. Under lateral load that wall would have a deformed shape not emulating that for a solid cast-in-place monolithic wall. Therefore the wall could not be designed using this provision.

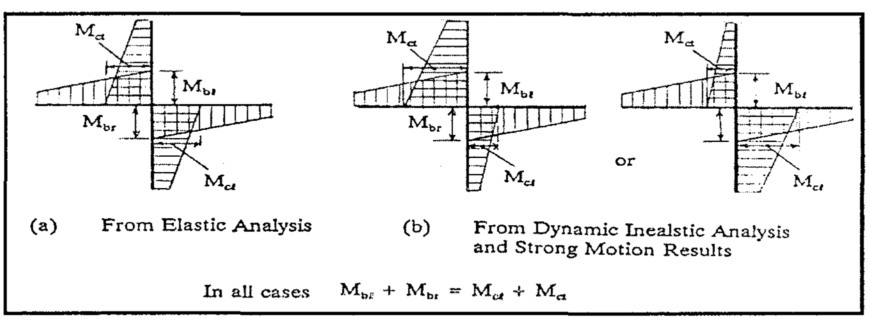


FIGURE C9.1.1-3 Moments at beam-to-column connections.

Sec. 21.11.3.1 in Sec. 9.1.1.12 recognizes that if the monolithic *wall* of Figure C9.1.1-4a, Part a, is composed of precast elements, as shown in Figure C9.1.1-4b, then the shear force acting on the connection at A-A can be limited by the shear capacity of the precast element above A-A, by the shear for slip along the connection, or by the probable connection moment capacity, M_{pr}. That moment corresponds to the value of H that causes a stress of $1.25f_y$ in the boundary reinforcement continuous across A-A. When the moment due to H causes a stress of $1.25f_y$ in the boundary reinforcement, the shear causing slip along the connection is less than if the steel stress was less than $1.25f_y$. The shear to cause slip decreases as the crack width increases. Only when the steel stress is limited to f_y can the shear *strength* be taken as that calculated by Sec. 11.7 of ACI 318. The probable shear *strength* is taken as that documented by Mueller (1989) and Wood (1990) for precast and monolithic *shear walls*, respectively.

The shear carrying mechanism of the monolithic *wall* of Figure C9.1.1-4a and that of the precast *wall* of Figure C9.1.1-4b are distinctly different when the overturning moment causes yielding of the boundary reinforcement and therefore opening of the horizontal connections. Lateral shears can then be transferred through compressed concrete only and the precast *wall* must be provided with horizontal reinforcement at the upper edge of the panel sufficient to balance the horizontal *component* of the force in the compression diagonal.

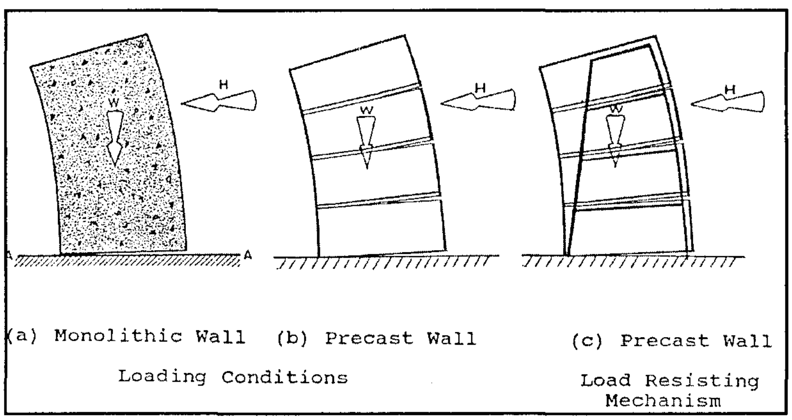


FIGURE C9.1.1-4 Conditions for walls.

Use of Prestressing Tendons: Sec. 9.1.1.5 defines conditions under which prestressing tendons can be used, in conjunction with deformed reinforcing bars, in frames resisting earthquake forces. As documented in Ishizuka and Hawkins (1987), if those conditions are met no modification is necessary to the R and C_d factors of Table 5.2.2 when prestressing is used. Satisfactory seismic performance can be obtained when prestressing amounts greater than those permitted by Sec. 9.1.1.5 are used. However, as documented by Park and Thompson (1977) and Thompson and Park (1980) and required by the combination of New Zealand Standards 3101:1982 and 4203:1992, ensuring that satisfactory performance requires modification of the R and C_d factors.

Structures Having Precast Concrete Gravity Load Carrying Systems: Sec. 9.1.1.4 defines conditions governing the design of structures, such as precast concrete parking garages, which have precast concrete gravity load carrying systems combined with either precast or cast-in-place seismic-force resisting systems. Further information on the background to the *Provisions* is provided in Ghosh et al., 1997. In the 1997 Provisions Sec. 21.2.1.7 in Sec. 9.1.1.5 required use of one of two methods with the first method differing from that specified in the 2000 Provisions and the second method being the same for both the 1997 and 2000 Provisions. The requirement in the first method of the 1997 Provisions that the span of the diaphragm or diaphragm segment between seismic-force resisting systems not exceed three times the width of the diaphragm or diaphragm segment has been deleted. The arbitrary 3:1 limit was imposed because of a lack of technical data. Based on analytical studies that requirement in the 2000 Provisions has been replaced by a requirement intended to ensure that the diaphragm remains elastic under the maximum design displacement and that there is sufficient chord reinforcement in the diaphragm to limit its maximum lateral deformation to 0.75 percent of the story height.

Structures Having Seismic-Force-Resisting Systems Utilizing Interconnected Precast Elements. Precast concrete seismic-force-resisting systems can be utilized only if: (1) substantiating experimental evidence of acceptable performance of that generic system has been demonstrated through cyclic tests on typical modules of that system; and (2) it is demonstrated through non-linear response history analysis using the evidence from those module tests that the system will perform satisfactorily under the Maximum Considered Earthquake Ground Motions. For special precast concrete moment frames substantiating experimental must satisfy the conditions specified in ACI Provisional Standard T1.1-99, (ACI Innovation Task Group 1 and Collaborators, 1999). Special precast concrete structural wall systems must satisfy similar conditions with the limiting drift ratio being a function of the height to width ratio for the wall as documented in Seo et al., 1998. The validity of the use of precast concrete seismic-force resisting systems has been demonstrated by the results of the recently completed PRESSS program (Priestley, 1991, 1996) and five story PRESSS building test (Priestley et al. 1999) and by analytical studies of precast/ prestressed concrete moment frames and structural walls (Cheok et al., 1998, El-Sheikh et al., 1999, Kurama et al. 1999).

Connections: Connections are classified into two types, X and Z in provision 21.11.6 in Sec. 9.1.1.12 in accordance with the ductilities achieved in acceptance tests on generic forms of those connections. Detailed information on performance of various connection types is contained in Schultz and Magana, 1996 and Pincheria et al., 1998.

9.2 ANCHORING TO CONCRETE:

9.2.1 Scope:

9.2.1.1: The *Provisions* are restricted in scope to structural anchors that transmit structural loads from attachments into concrete members. The levels of safety defined by the combinations of load factors and ϕ factors are appropriate for structural applications. Other standards can require more stringent safety levels during temporary handling.

9.2.1.2: The wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of the Provisions. Bonded anchors, held in place by grout, epoxy, resins, or other chemicals are widely used and can perform adequately. However, at this time such anchors are outside the scope of the *Provisions*.

9.2.1.3: Typical cast-in headed studs and headed bolts with geometries consistent with ANSI/ASME B1.1 (1989), B18.2.1 (1996), and B18.2.6 (1996) have been tested and have proven to behave predictably, so calculated pullout values are acceptable. Post-installed anchors do not have predictable pullout capacities, and therefore are required to be tested.

9.2.1.4: Post-installed fasteners designed using the Provisions must first be qualified in accordance with a comprehensive set of tests. The tests shall include reference tests, reliability tests, and service-condition tests. The reference tests should establish basic anchor performance and capacity for failure modes, including concrete breakout, steel rupture, or pullout. The reliability tests should establish fastener performance under adverse installation conditions expected to be found under field conditions, and should provide the information necessary to establish the ϕ factors to be used in Sec. 9.2.4.4 or 9.2.4.5. Service-condition tests should determine if the fasteners are appropriate for use under these design provisions with respect to edge distance, fastener spacing, shear capacity, pryout, splitting near an edge, and seismic capacity.

Standards for qualification tests with these attributes are under preparation in ACI (ACI 355.2) and will be subsequently processed in ASTM (Z5819Z). These documents contain requirements for testing and certification of post-installed fasteners for both cracked and uncracked concrete applications including qualification for use in seismic applications. Anchor prequalification tests should require that anchors qualified for use in cracked concrete perform well in cracks whose width is consistent with that intended by the requirements of Sec. 10.6.4 of ACI 318.

9.2.1.5: The exclusion from the scope of load applications producing high cycle fatigue or extremely short duration impact (such as blast or shock wave) are not meant to exclude seismic load effects. Sec. 9.2.3.3 presents additional requirements for design when seismic loads are included.

9.2.2 Notations and Definitions:

9.2.2.1 Notations:

A_{se} = The effective cross-sectional area of an anchor should be provided by the manufacturer of expansion anchors with reduced cross-sectional area for the expansion mechanism. For threaded bolts, ANSI/ASME B1.1 (1989) defines A_{se} as:

$$A_{se} = \frac{\pi}{4}\left(d_0 - \frac{0.9743}{n_t}\right)^2 \qquad \text{(C9.2.2.1)}$$

where n_t =is the number of threads per inch.

e_n = Actual eccentricity of a normal force on an attachment

h_{ef} = Effective embedment depths for a variety of anchor types are shown in Figure C9.2.2.1.

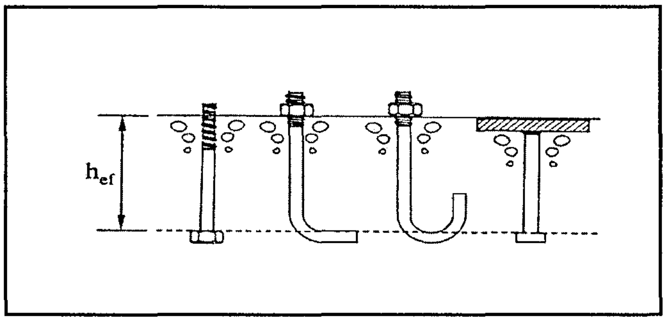


FIGURE C9.2.2.1 Types of cast-in-place anchors.

9.2.2.2 Definitions:

5 Percent Fractile – The determination of the coefficient K associated with the 5 percent fractile, $\bar{x} - \sigma$, $\bar{x} - K\sigma$, depends on the number of tests, n, used to compute $\bar{x}$ and σ. Values of K range, for example, from 1.645 for $n = \infty$ to 2.010 for $n = 40$ and 2.568 for $n = 10$. With this definition of the 5 percent fractile, the nominal strength in Sec. 9.2.4.2 is the same as the characteristic strength in the anchor prequalification tests.

9.2.3 General Requirements:

9.2.3.1: When the strength of an anchor group is governed by breakage of the concrete, the behavior is brittle and there is limited redistribution of the forces between the highly stressed and less stressed anchors. In this case, the theory of elasticity is required to be used assuming the attachment that distributes loads to the anchors is sufficiently stiff. The forces in the anchors are considered to be proportional to the external load and its distance from the neutral axis of the anchor group.

If anchor strength is governed by ductile yielding of the anchor steel, significant redistribution of anchor forces can occur. In this case, an analysis assuming the theory of elasticity will be conservative. The works by Cook and Klingner (Feb. 1992), Cook and Klingner (June 1992), and Lotze and Klingner (1997) discuss non-linear analysis, using theory of plasticity, for the determination of the capacities of ductile anchor groups.

9.2.3.3: Post-installed structural anchors are required to be qualified for moderate or high seismic risk zone usage by passing anchor prequalification simulated seismic tests. In addition, the design of anchors in zones of moderate or high seismic risk is based on a more conservative approach by the introduction of a 0.75 factor on the design strength ϕN_n and ϕV_n, and by requiring ductile failures. Alternatively, a higher value of anchor strength can be used if the attachment being fastened is designed to ensure ductile yielding of the attachment at a load well below the minimum probable anchor strength.

For an anchor to be acceptable in seismic loading situations the system is required to have adequate ductility. The anchor is required to demonstrate the capacity to undergo large displacements through several cycles as specified in the anchor prequalification seismic simulation tests. If the anchor cannot meet these requirements, then the attachment is required to be designed so as to yield at a load well below the anchor capacity. In designing attachments for adequate ductility, the ratio of yield to ultimate load capacity should be considered. A connection element could yield only to result in a secondary failure as one or more elements strain harden if the ultimate load capacity is excessive when compared to the yield capacity.

Under seismic conditions, the direction of shear loading may not be predictable. The full shear load should be assumed in any direction for a safe design.

9.2.3.5: A limited number of tests of cast-in and post-installed anchors in high-strength concrete (see Primavera, Pinelli, and Kalajian (1997)) indicate that the design procedures contained in the Provisions become unconservative, particularly for cast-in anchors, at f'_c = 11,000 to 12,000 psi. Until further test results are available, an upper limit of f'_c = 10,000 psi was imposed in the design of cast-in anchors. This is consistent with Chapters 11 and 12 of ACI 318. The anchor prequalification standard does not require testing of post-installed anchors in concrete with $f'_c >$ 8,000 psi since some post-installed anchors may have difficulty expanding in very high strength concretes. Because of this, f'_c is limited to 8000 psi in the design of post-installed anchors.

9.2.4 General Requirements for Strength of Structural Anchors:

9.2.4.1: This section provides the requirements for establishing the strength of anchors to concrete. The various types of steel and concrete failure modes for anchors are shown in Figures C9.2.4.1-1 and C9.2.4.1-2. Comprehensive discussions of anchor failure modes are included in *Design of Fastenings in Concrete* (1997), Fuchs, Eligehausen, Breen (1995), and Eligehausen and Balogh (1995). Any model that complies with the requirements of Sec. 9.2.4.2 and 9.2.4.3 can be used to establish the concrete related strengths. For anchors such as headed bolts, headed studs and post-installed anchors, the concrete breakout design method of Sec. 9.2.5.2 and 9.2.6.2 is acceptable. The anchor strength is also dependent on the pullout strength of Sec. 9.2.5.3, the side-face blowout strength of Sec. 9.2.5.4 and the minimum spacings and edge distances of Sec. 9.2.8. The design of anchors for tension recognizes that the strength of anchors is sensitive to appropriate installation; installation requirements are included in Sec. 9.2.9. Some post-installed anchors are less sensitive to installation errors and tolerances. This is reflected in varied ϕ factors based on the assessment criteria of the anchor prequalification tests.

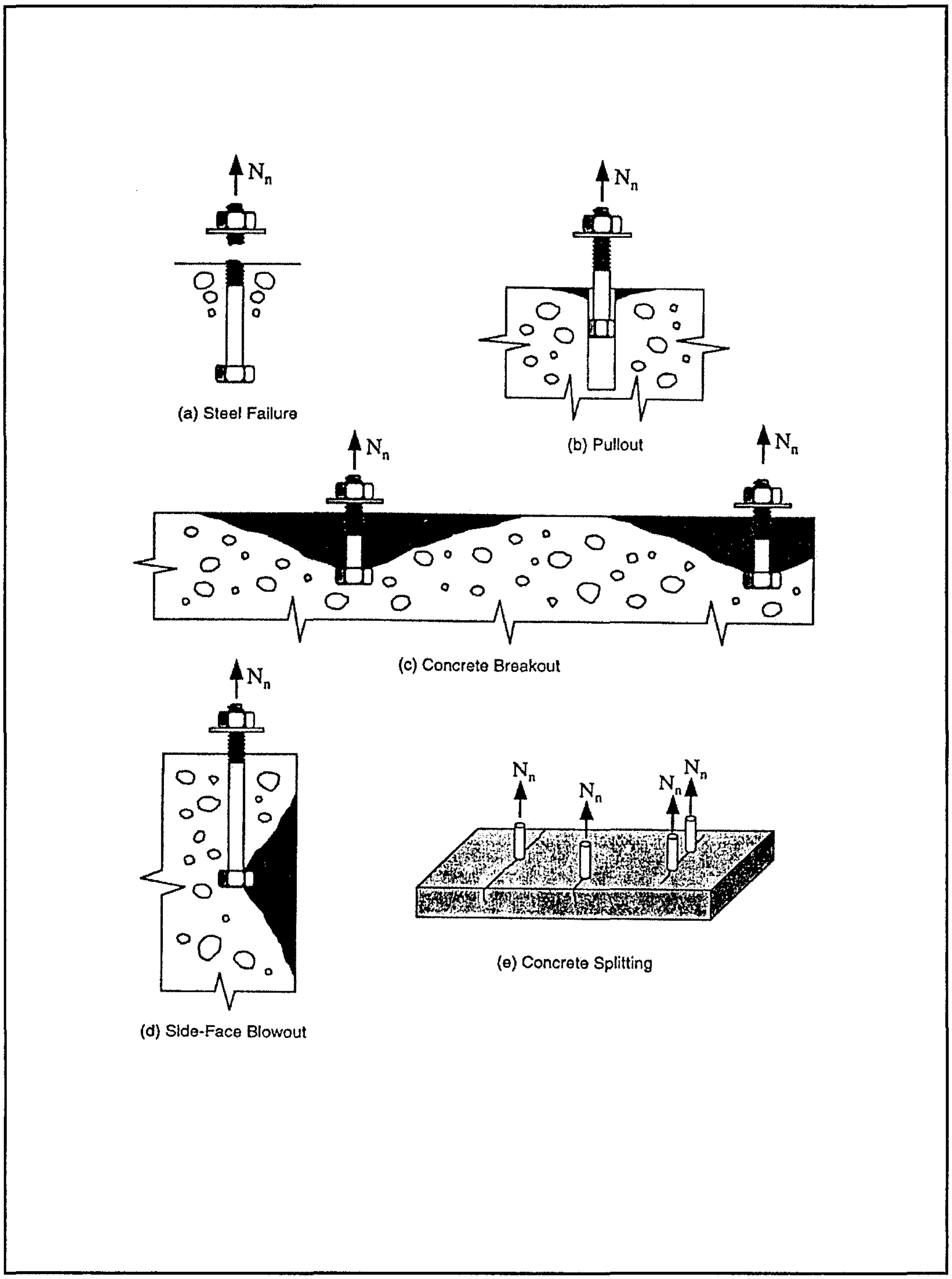


FIGURE C9.2.4.1-1 Failure modes for anchors under tensile loading.

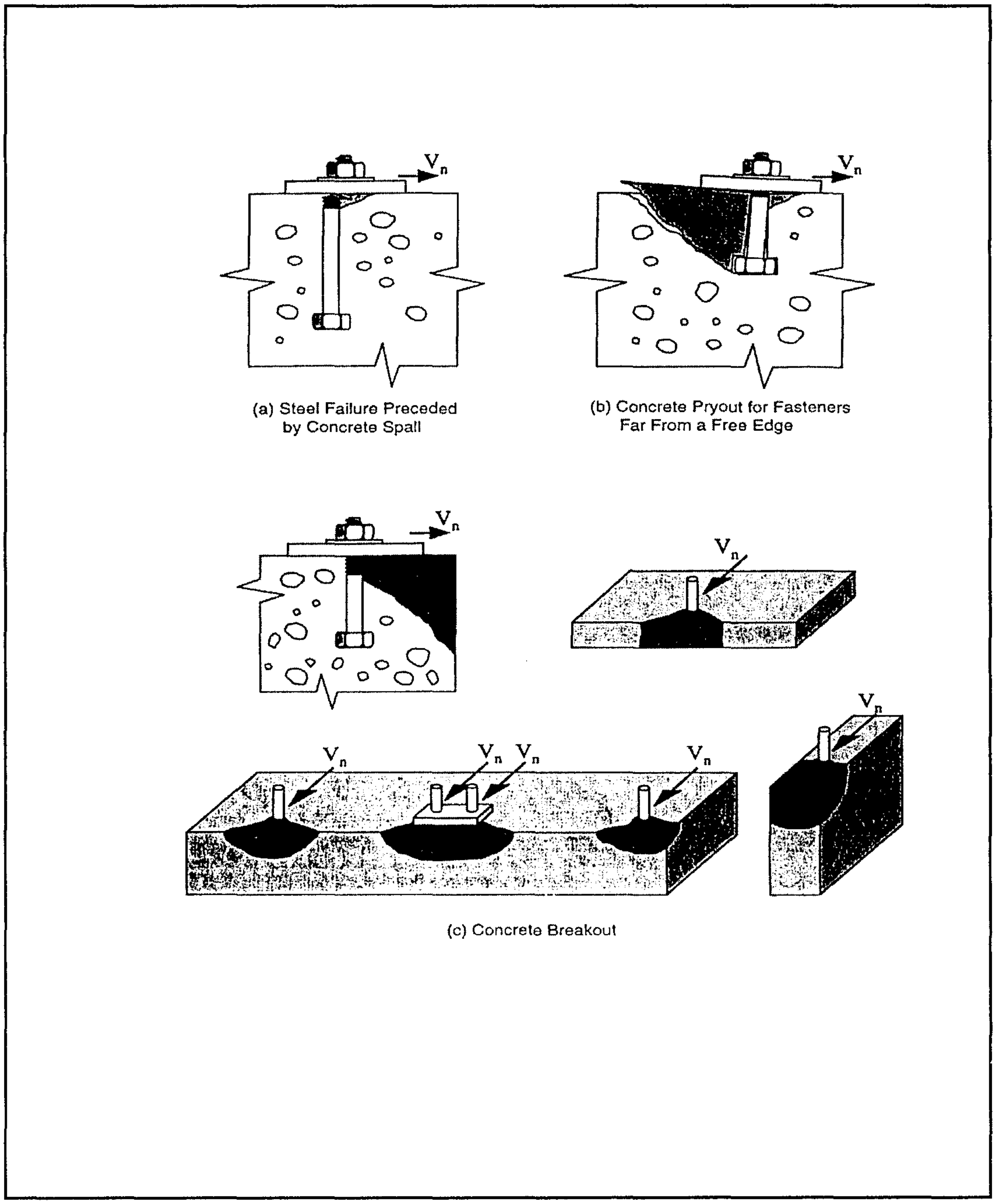


FIGURE C9.2.4.1-2 Failure modes for anchors under shear loading.

Test procedures can also be used to determine the single-anchor breakout strength in tension and in shear. However, the test results are required to be evaluated on a basis statistically equivalent to that used to select the values for the concrete breakout method "considered to satisfy" provisions of Sec. 9.2.4.2. The basic strength cannot be taken greater than the 5 percent fractile.

The number of tests has to be sufficient for statistical validity and should be considered in the determination of the 5 percent fractile.

9.2.4.2 and 9.2.4.3: These sections establish the performance factors for which anchor design models are required to be verified. Many possible design approaches exist and the user is always permitted to "design by test" using Sec. 9.2.4.2 as long as sufficient data are available to verify the model.

9.2.4.2.1: The addition of supplementary reinforcement in the direction of the load, confining reinforcement, or both, can greatly enhance the strength and ductility of the anchor connection. Such enhancement is practical with cast-in anchors such as those used in precast sections.

The shear strength of headed anchors located near the edge of a member can be significantly increased with appropriate supplementary reinforcement. *Design of Fastenings in Concrete* (1997), *Fastenings in Concrete and Masonry Structures, State of the Art Report* (1994)and Klingner, Mendonca, and Malik (1982) provide substantial information on design of such reinforcement. The effect of such supplementary reinforcement is not included in the anchor prequalification tests or in the concrete breakout calculation method of Sec. 9.2.5.2 and 9.2.6.2. The designer has to rely on other test data and design theories in order to include the effects of supplementary reinforcement.

For anchors exceeding the limitations of Sec. 9.2.4.2.2, for situations where geometric restrictions limit breakout capacity, or both, reinforcement proportioned to resist the total load, oriented in the direction of load, within the breakout prism and fully anchored on both sides of the breakout planes, may be provided instead of calculating breakout capacity.

The breakout strength of an unreinforced connection can be taken as an indication of the load at which significant cracking will occur. Such cracking can represent a serviceability problem if not controlled. (See Sec. 9.2.6.2.1)

9.2.4.2.2: The method for concrete breakout design included as "considered to satisfy" Sec. 9.2.4.2 was developed from the Concrete Capacity Design (CCD) Method (see Fuchs, Eligehausen and Breen (1995), and Eligehausen and Balogh (1995), which was an adaptation of the κ Method (see Eligehausen, Fuchs and Mayer (1987), and (Eligehausen and Fuchs (1988), and is considered to be accurate, relatively easy to apply, and capable of extension to irregular layouts. The CCD Method predicts the load-bearing capacity of an anchor or group of anchors by using a basic equation for tension or for shear for a single anchor in cracked concrete, and multiplying by factors which account for the number of anchors, edge distance, spacing, eccentricity and absence of cracking. The limitations on anchor size and embedment length are based on the current range of test data.

The breakout strength calculations are based on a model suggested in the κ Method. It is consistent with a breakout prism angle of approximately 35 degrees (Figures C.9.2.4.2.2-1and C9.2.4.2.2-2).

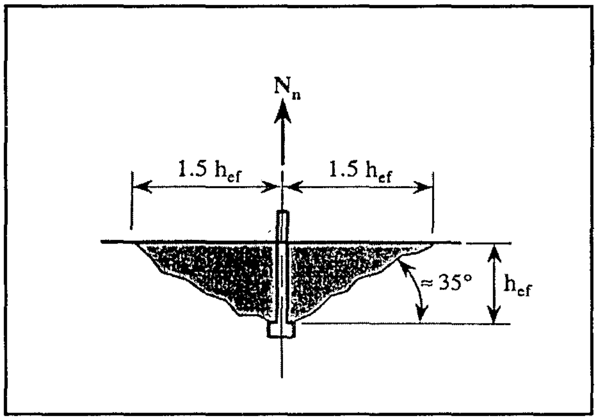


FIGURE C9.2.4.2.2-1 Breakout cone for tension.

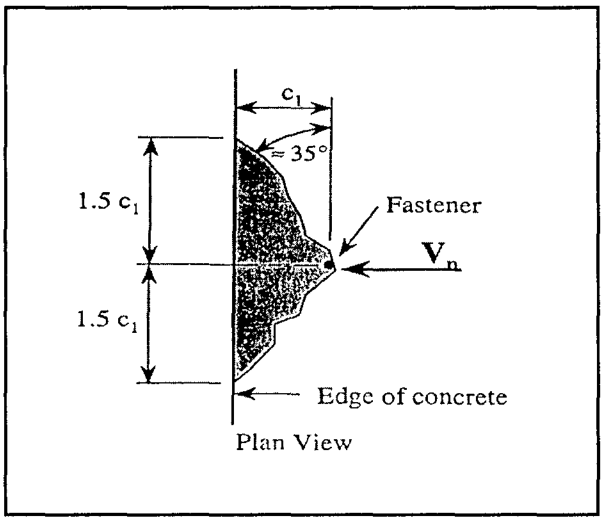


FIGURE C9.2.4.2.2-2 Breakout cone for shear.

9.2.4.4: The ϕ factor for failure of ductile elements is indicative of less variability in steel tension failures than concrete breakout failures, and the greater amount of warning with a ductile failure. It is acceptable to have a ductile failure of a steel element in the attachment if the attachment is designed so that it will undergo ductile yielding at a load level no greater than 75 percent of the minimum anchor design strength (See Sec. 9.2.3.3.4). For anchors governed by the more brittle concrete breakout or blowout failure, two conditions are recognized. If supplementary reinforcement is provided to tie the failure prism into the structural member (Condition A), more ductility is present than in the case where such supplementary reinforcement is not present (Condition B). Design of supplementary reinforcement is discussed in Sec. 9.2.4.2.1 and the References by Primavera, Pinelli, and Kalajian (1997), Cook and Klingner (June 1992), and ACI Committee 349-85. Even though the ϕ factor for plain concrete uses a value of 0.65, the basic factor for brittle failures ($\phi = 0.75$) has been chosen based on the results of probabilistic studies (Farrow and Klingner (1995)) that indicated that for anchoring to concrete the use of $\phi = 0.65$ with mean values of concrete-controlled failures produced adequate safety levels. However, the nominal resistance expressions used in the *Provisions* and in the test requirements are the 5 percent fractiles. Thus, the $\phi = 0.65$ value would be overly conservative. Comparison with other design procedures and probabilistic studies by Farrow and Klingner (1995) indicated that the choice of $\phi = 0.75$ was justified. For applications with supplementary reinforcement and more ductile failures (Condition A), the ϕ factors are increased. The value of $\phi = 0.85$ is compatible with the level of safety for shear failures in concrete beams, and has been recommended by the *PCI Design Handbook* (1992) and ACI 349-85 .

The anchor prequalification tests for sensitivity to installation procedures determine the category appropriate for a particular anchoring device. In the prequalification tests, the effects of variability in anchor torque during installation, tolerance on drilled hole size, energy level used in setting anchors, and lateral contact with reinforcement are considered. The three categories of acceptable post-installed anchors are:

Category 1 - systems with high installation safety

Category 2 - systems with medium installation safety

Category 3 - systems with lower but still acceptable installation safety

The capacities of anchors under shear loads are not as sensitive to installation errors and tolerances. Therefore, for shear calculations of all anchors $\phi = 0.85$ for Condition A and $\phi = 0.75$ for Condition B.

9.2.5 Design Requirements for Tensile Loading:

9.2.5.2 Concrete Breakout Strength of Anchor in Tension:

9.2.5.2.1: The effects of multiple anchors, spacing of anchors, and edge distance on the nominal concrete breakout strength in tension are included by applying the modification factors A_N / A_{No} and y_2 in Eq. 9.2.5.2.1-1 or -2.

Figure C.9.2.5.2.1-1 shows A_{No} and the development of Eq. 9.2.5.2.1-3. A_{No} is the maximum projected area for a single anchor. Figure C9.2.5.2.1-2 shows examples of the projected areas for various single anchor and multiple anchor arrangements. Because A_N is the total projected area for a group of anchors, and A_{No} is the area for a single anchor, there is no need to include *n*, the number of anchors, in Eq. 9.2.5.2.1-1 or 9.2.5.2.1-2. If anchor groups are positioned in such a way that their projected areas overlap, the value of A_N is required to be reduced accordingly.

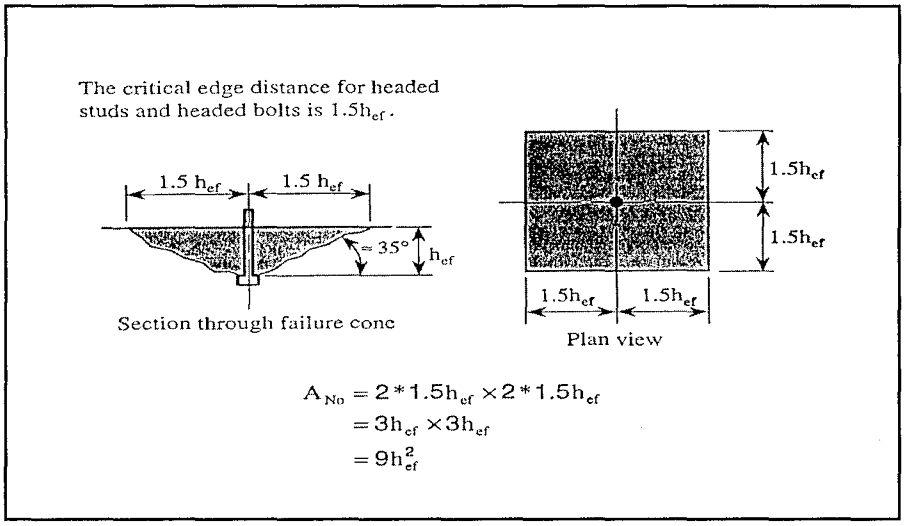


FIGURE C9.2.5.2.1-1 Calculation of A_{N0}.

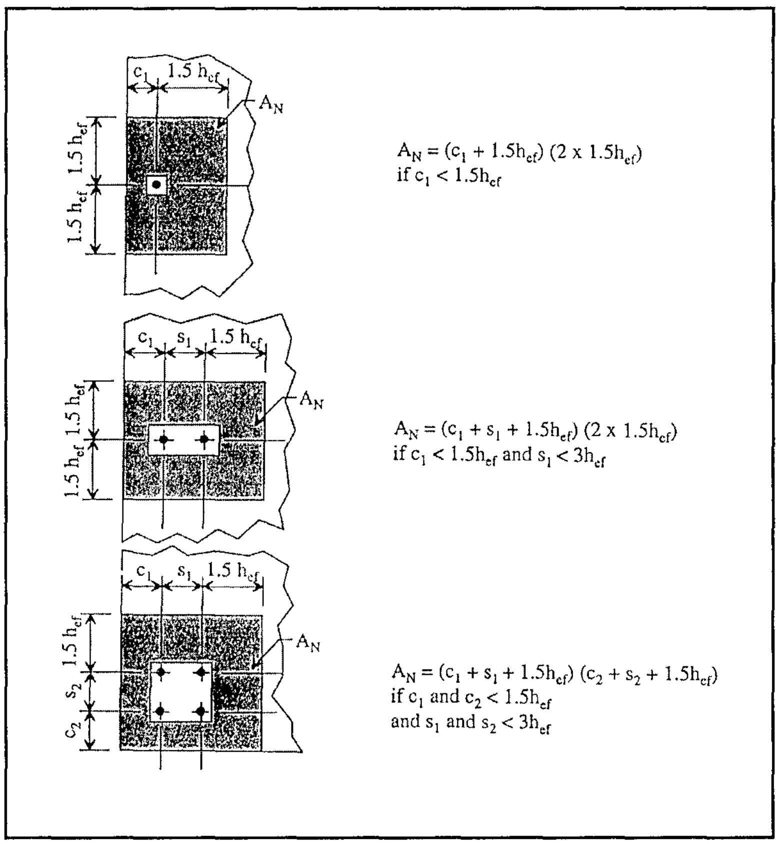


FIGURE C9.2.5.2.1-2 Projected areas for single anchors and groups of anchors.

9.2.5.2.2: The basic equation for anchor capacity was derived (Fuchs, Eligehausen, and Breen (1991), Eligehausen and Balogh (1995), Fastenings to Concrete and Masonry Structures (1994), and Eligehausen and Fuchs (1998)) assuming a concrete failure prism with an angle of about 35 degrees, and considering fracture mechanics concepts.

The values of k were determined from a large database of test results in uncracked concrete (Fuchs, Eligehausen and Breen (1995) as the 5 percent fractile. The values were adjusted to corresponding k values for cracked concrete (Eligehausen and Balogh (1995) and Zhang (1997)). For anchors with a deep embedment (h_{ef} > 11 in.) some test evidence indicates the use of $h_{ef}^{1.5}$ can be overly conservative for some cases. Often such tests have been with selected aggregates for special applications. An alternate expression (Eq. 9.2.5.2.2-2) is provided using $h_{ef}^{5/3}$ for evaluation of cast-in anchors with 11 in. < h_{ef} < 25 in. The limit of 25 in. corresponds to the upper range of test data. This expression can also be appropriate for some undercut post-installed anchors. However, Sec. 9.2.4.2 should be used with test results to justify such applications.

9.2.5.2.3: For anchors influenced by three or more edges where any edge distance is less than 1.5 h_{ef}, the tensile breakout strength computed by the ordinary CCD method, which is the basis for Eq. 9.2.5.2.2-1, gives misleading results. This occurs because the ordinary definitions of A_N/A_{No} do not correctly reflect the edge effects. However, if the value of h_{ef} is limited to $c_{max}/1.5$, where c_{max} is the largest of the influencing edge distances that are less than or equal to the actual $1.5h_{ef}$, this problem is corrected. As shown by Lutz (1995), this limiting value of h_{ef} is to be used in Eq. 9.2.5.2.1-3, 9.2.5.2.2-1, 9.2.5.2.4, and 9.2.5.2.5-1 or -2. This approach is best understood when applied to an actual case. Figure C9.2.5.2.3 shows how the failure surface has the same area for any embedment beyond the proposed limit on h_{ef} (taken as h'_{ef} in the figure). In this example, the proposed limit on the value of h_{ef} to be used in the computations where $h_{ef} = c_{max}/1.5$, results in $h_{ef} = h'_{ef} = 4$ in./1.5 = 2.67 in. This would be the proper value to be used for h_{ef} in computing the resistance, for this example, even if the actual embedment depth is larger.

9.2.5.2.4: Figure C9.2.5.2.4-1 shows dimension $e'_N = e_N$ for a group of anchors that are all in tension but that have a resultant force eccentric with respect to the centroid of the anchor group. Groups of anchors can be loaded in such a way that only some of the anchors are in tension (Figure C9.2.5.2.4-2). In this case, only the anchors in tension are to be considered in the determination of e'_N. The anchor loading has to be determined as the resultant anchor tension at an eccentricity with respect to the center of gravity of the anchors in tension. Eq. 9.2.5.2.4 is limited to cases where $e'_N < s/2$ to alert the designer that all anchors may not be in tension.

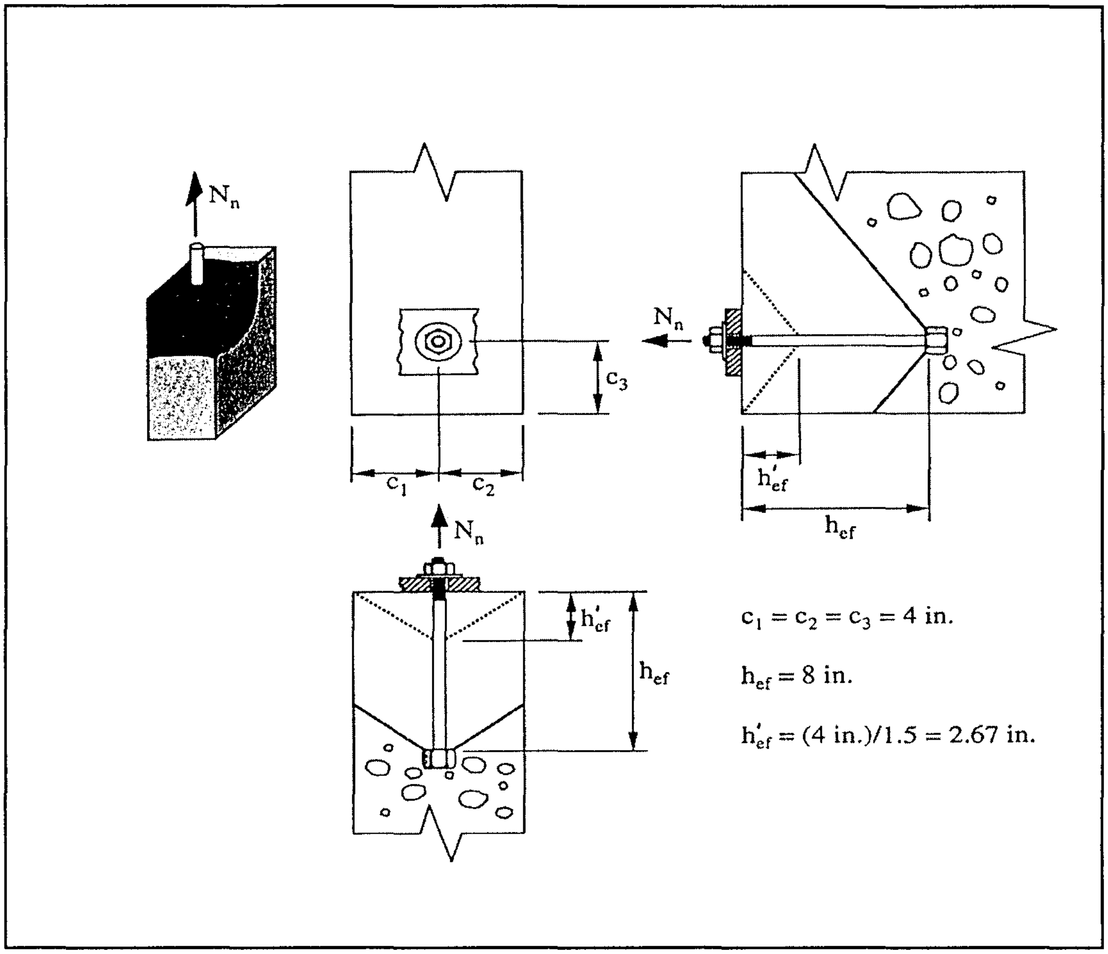


FIGURE C9.2.5.2.3 Failure surfaces in narrow members for different embedment depths.

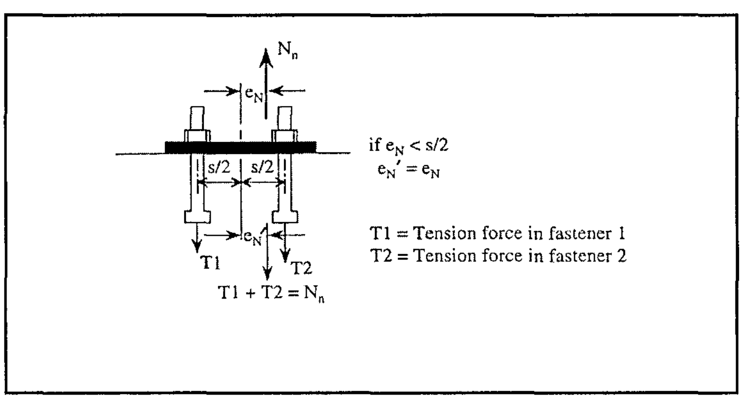


FIGURE C9.2.5.2.4-1 Definition of dimension e_N' when all anchors in a group are in tension.

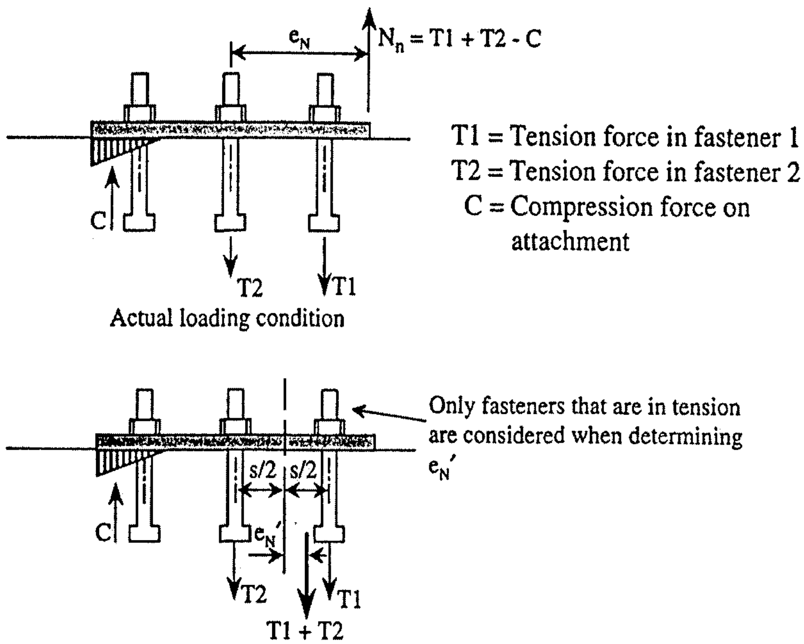


FIGURE C9.2.5.2.4-2 Determination of e_N' for anchor group with only some anchors in tension.

9.2.5.2.5: If anchors are located close to an edge so that there is not enough space for a complete breakout prism to develop, the load bearing capacity of the anchor is further reduced beyond that reflected in A_N/A_{No}. If the smallest side cover distance is greater than 1.5 h_{ef}, a complete prism can form and there is no reduction ($Y_2 = 1$). If the side cover is less than 1.5 h_{ef}, the factor, Y_2, is required to adjust for the edge effect (Lotze and Klingner (1997).

9.2.5.2.6: Post-installed and cast-in anchors that have not met the requirements for use in cracked concrete according to the anchor prequalification tests should be used in uncracked regions only. The analysis for the determination of crack formation should include the effects of restrained shrinkage.

9.2.5.2.7: The anchor prequalification tests require that anchors in cracked concrete zones perform well in a crack that is 0.012 in. wide. If wider cracks are expected, confining reinforcement to control the crack width to about 0.012 in. should be provided.

9.2.5.3 Pullout Strength of Anchors in Tension:

9.2.5.3.3: The pullout strength in tension of headed studs or headed bolts can be increased by provision of confining reinforcement, such as closely spaced spirals, throughout the head region. This increase can be demonstrated by tests.

9.2.5.3.4: Eq. 9.2.5.3.4 corresponds to the load at which the concrete under the anchor head begins to crush. (*Design of Fastenings in Concrete* (1997) and ACI 349-85). It is not the load required to pull the anchor completely out of the concrete, so the equation contains no term relating to embedment depth. The designer should be aware that local crushing under the head bearing region will greatly reduce the stiffness of the connection, and generally will be the beginning of a pullout failure.

9.2.5.3.5: Eq. 9.2.5.3.5 for J-bolts and L-bolts was developed by Lutz based on the results of work by Kuhn and Shaikh (1996). Reliance is placed on the bearing *component* only, neglecting any frictional *component* since local crushing under the head will greatly reduce the stiffness of the connection, and generally will be the beginning of pullout failure.

9.2.5.4 Concrete Side-Face Blowout Strength of Anchor in Tension: The design requirements for side-face blowout are based on the recommendations of Furche and Eligehausen (1991). These requirements are applicable to headed anchors that usually are cast-in anchors. Splitting during installation rather than sideface blowout generally governs post-installed anchors, and is evaluated by the anchor prequalification tests.

9.2.6 Design Requirements for Shear Loading:

9.2.6.2 Concrete Breakout Strength of Anchors in Shear:

9.2.6.2.1: The shear strength equations were developed from the CCD method. They assume a breakout cone angle of approximately 35 degrees Figure C9.2.4.2.2-2, and consider fracture mechanics theory. The effects of multiple anchors, spacing of anchors, edge distance and thickness of the concrete member on nominal concrete breakout strength in shear are included by applying the reduction factor A_V/A_{Vo} and ι_5 in Eq. 9.2.6.2.1-1 or -2. For anchors far from the edge, Sec. 9.2.6.2 usually will not govern. For these cases, Sec. 9.2.6.1 and Sec. 9.2.6.3 often govern.

Figure C9.2.6.2.1-1 shows A_{Vo} and the development of Eq. 9.2.6.2.1-3. A_{Vo} is the maximum projected area for a single anchor that approximates the surface area of the full breakout prism or cone for an anchor unaffected by edge distance, spacing or depth of member. Figure C9.2.6.2.1-2 shows examples of the projected areas for various single anchor and multiple anchor arrangements. A_V approximates the full surface area of the breakout cone for the particular arrangement of anchors. Since A_V is the total projected area for a group of anchors, and A_{Vo} is the area for a single anchor, there is no need to include the number of anchors in the equation.

The assumption shown in Figure C9.2.6.2.1-2 with the case for two anchors perpendicular to the edge is a conservative interpretation of the distribution of the shear force on an elastic basis. If the anchors are welded to a common plate, when the anchor nearest the front edge begins to form a failure cone, shear load would be transferred to the stiffer and stronger rear anchor. For cases where nominal strength is not controlled by ductile steel elements, ACI Committee 318 has specified in Sec. 9.2.3.1 that load effects be determined by elastic analysis. It has been suggested in the *PCI Design Handbook* approach (1992) that the increased capacity of the anchors away from the edge be considered. Because this is a reasonable approach assuming that the anchors are spaced far enough apart so that the shear failure surfaces do not intersect (*Fastenings to Concrete and Masonry Structures* (1994)), Sec. 9.2.6.2 allows such a procedure. If the failure surfaces do not intersect, as would generally occur if the anchor spacing, s, is equal to or greater than $1.5c_l$, then after formation of the near-edge failure surface, the higher capacity of the farther anchor would resist most of the load. As shown in the bottom example in Figure C9.2.6.2.1-2, it would be appropriate to consider the full shear capacity to be provided by this anchor with its much larger resisting failure surface. No contribution of the anchor near the edge is then considered. It would be advisable to check the near-edge anchor condition to preclude undesirable cracking at service load conditions. Further discussion of design for multiple anchors is given *in Design of Fastenings in Concrete* (1997).

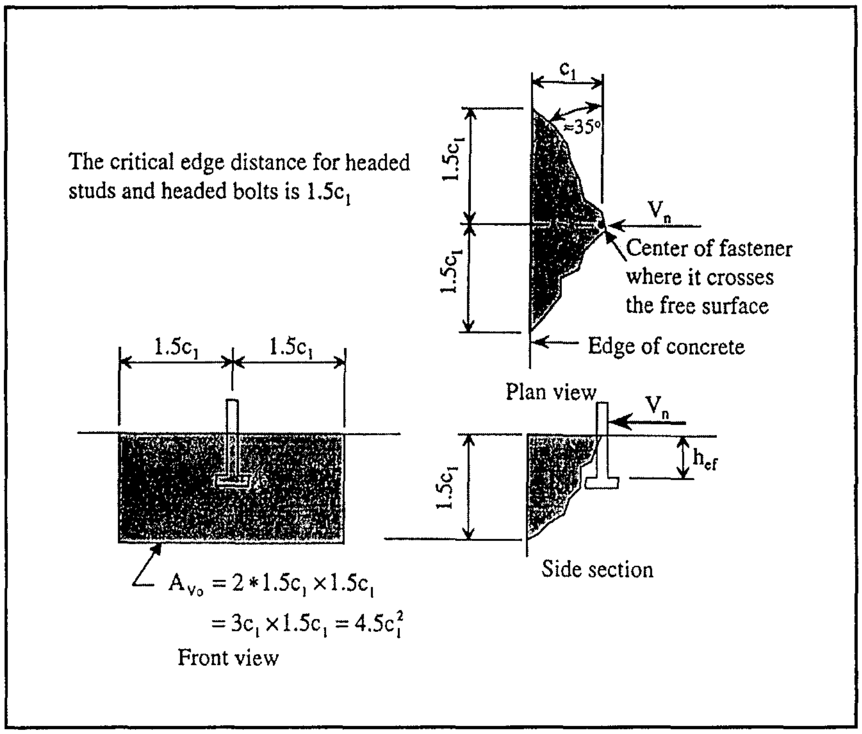


FIGURE C9.2.6.2.1-1 Calculation of A_{V0}.

For the case of anchors near a corner subjected to a shear force with *components* normal to each edge, a satisfactory solution is to independently check the connection for each *component* of the shear force. Other specialized cases, such as the shear resistance of anchor groups where all anchors do not have the same edge distance, are treated in *Fastenings to Concrete and Masonry Structures* (1994).

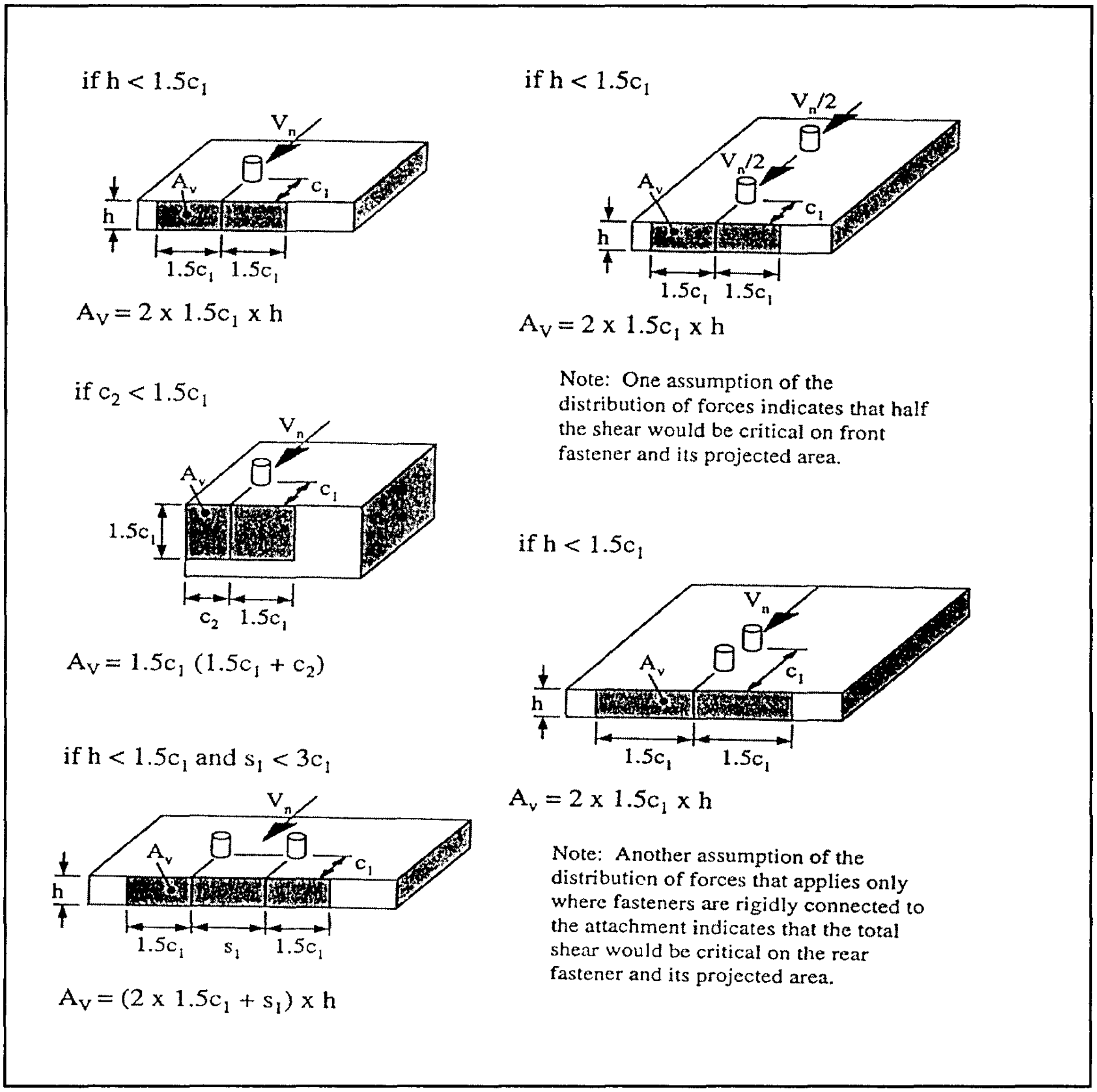


FIGURE C9.2.6.2.1-2 Projected areas for single anchor and groups of anchors.

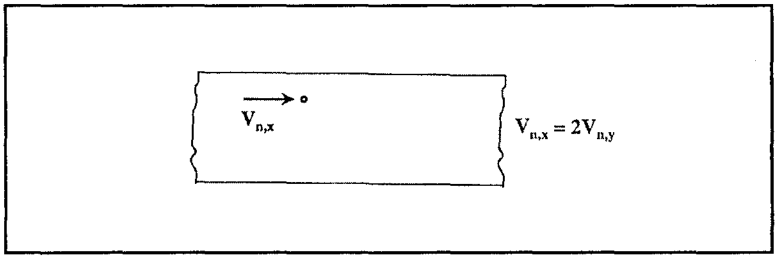


FIGURE C9.2.6.2.1-3 Shear force parallel to an edge.

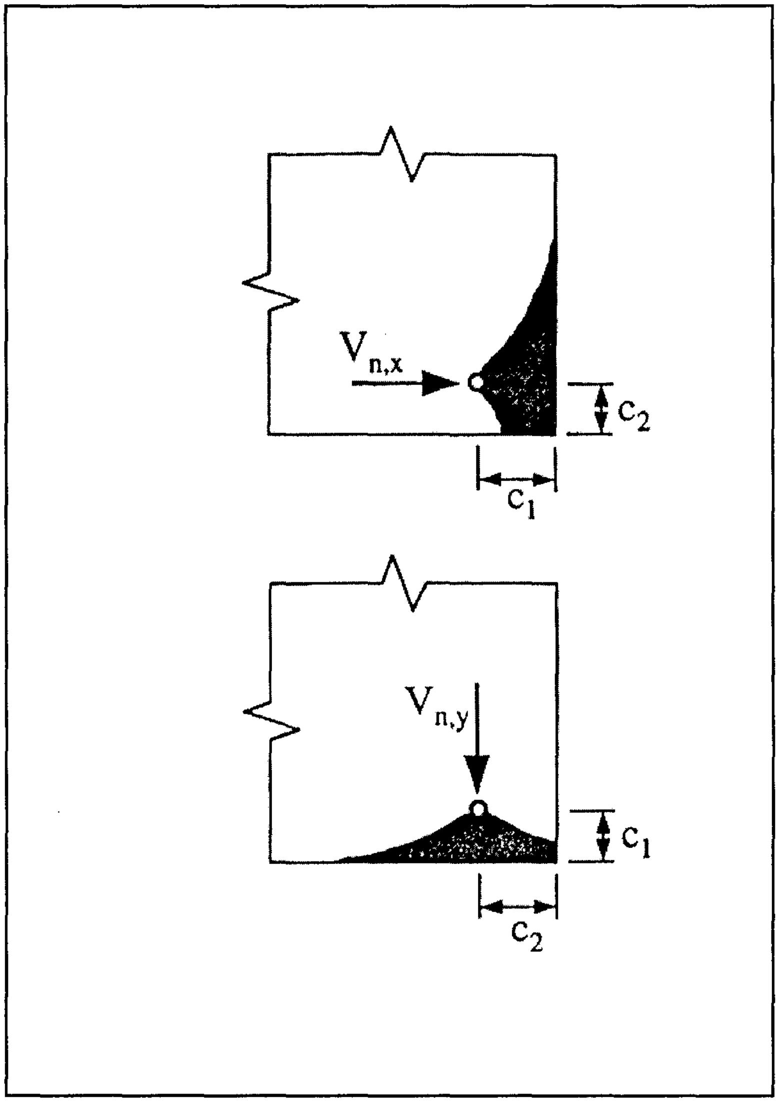


FIGURE C9.2.6.2.1-4 Anchors near a corner.

The detailed provisions of Sec. 9.2.6.2.1 (a) apply to the case of shear force directed towards an edge. When the shear force is directed away from the edge, the strength will usually be governed by Sec. 9.2.6.1 or 9.2.6.3.

The case of shear force parallel to an edge (Sec. 9.2.6.2.1b) is shown in Figure C9.2.6.2.1-3. A special case can arise with shear force parallel to the edge near a corner. Take the example of a single anchor near a corner (Figure C9.2.6.2.1-4). If the edge distance to the side c_2 is 40 percent or more of the distance c_1 in the direction of the load, the shear strength parallel to that edge can be computed directly from Eq. 9.2.6.2.1-1 or -2 using c_1 in the direction of the load.

9.2.6.2.2: Like the concrete breakout tensile capacity, the concrete breakout shear capacity does not increase with the failure surface, which is proportional to c_1^2. Instead the capacity increases proportionally to $c_1^{1.5}$, due to the size effect. The capacity is also influenced by the anchor stiffness and the anchor diameter D. (see diameter. (See Fuchs, Eligehausen, and Breen (1995), Eligehausen and Balogh (1995), *Fastenings to Concrete and Masonry Structures* (1994), and Eligehausen and Fuchs (1988).

The constant 7 in the shear strength equation was determined from test data reported in the article by Fuchs, Eligehausen, and Breen (1995) as the 5 percent fractile adjusted for cracking.

9.2.6.2.3: For the special case of cast-in headed bolts rigidly welded to an attachment, test data (Wong (1988) [1988] and Shaikh and Yi (1985) [1985]) show that somewhat higher shear capacity exists, possibly due to the stiff welding connection clamping the bolt more effectively than an attachment with a anchor gap. Because of this, the basic shear value for such anchors is increased. Limits are imposed to ensure sufficient rigidity. The design of supplementary reinforcement is discussed in *Design of Fastenings in Concrete* (1997*), Fastenings to Concrete and Masonry Structures* (1994) and Klingner, Mendonca, and Malik (1982).

9.2.6.2.4: For anchors influenced by three or more edges where any edge distance is less than $1.5c_1$, the shear breakout strength computed by the basic CCD Method, which is the basis for Eq. 9.2.6.2.2 or 9.2.6.2.3, gives safe but misleading results. These special cases were studied for the κ method (Eligehausen and Fuchs (1988)) and the problem was pointed out by Lutz (1995).

Similar to the approach used for tensile breakouts in Sec. 9.2.5.2.3, a correct evaluation of the

capacity is determined if the value of c_1 to be used in Eq. 9.2.6.2.1-3, 9.2.6.2.2 or 9.2.6.2.3, 9.2.6.2.5 and 9.2.6.2.6-1 or -2 is limited to $h/1.5$.

9.2.6.2.5: This section provides a modification factor for an eccentric shear force towards an edge on a group of anchors. If the shear load originates above the plane of the concrete surface, the shear should first be resolved as a shear in the plane of the concrete surface, with a moment that can or cannot also cause tension in the anchors, depending on the normal force. Figure C9.2.6.2.5 defines the term e'_v for calculating the Ψ_5 modification factor that accounts for the fact that more shear is applied on one anchor than the other, tending to split the concrete near an edge. If $e'_v > s/2$, the CCD procedure is not applicable.

9.2.6.2.6: Figure C9.2.6.2.6 shows the dimension c_2 for the Ψ_6 calculation.

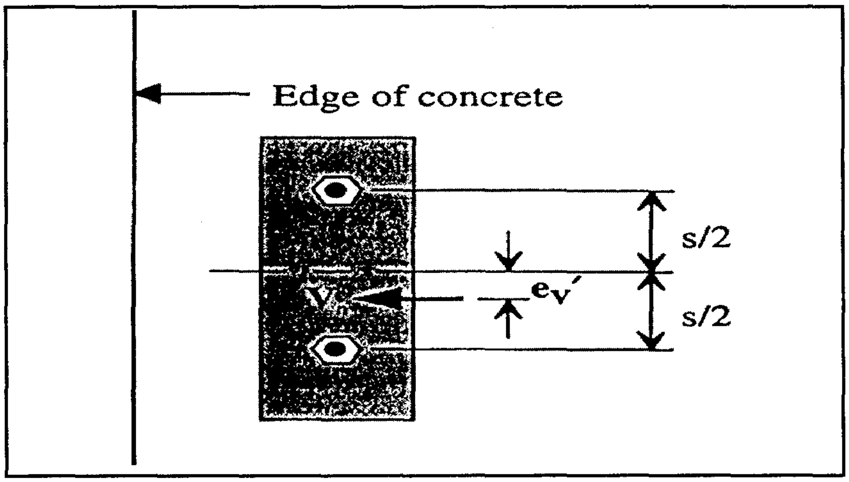


FIGURE C9.2.6.2.5 Definition of dimension e_V'.

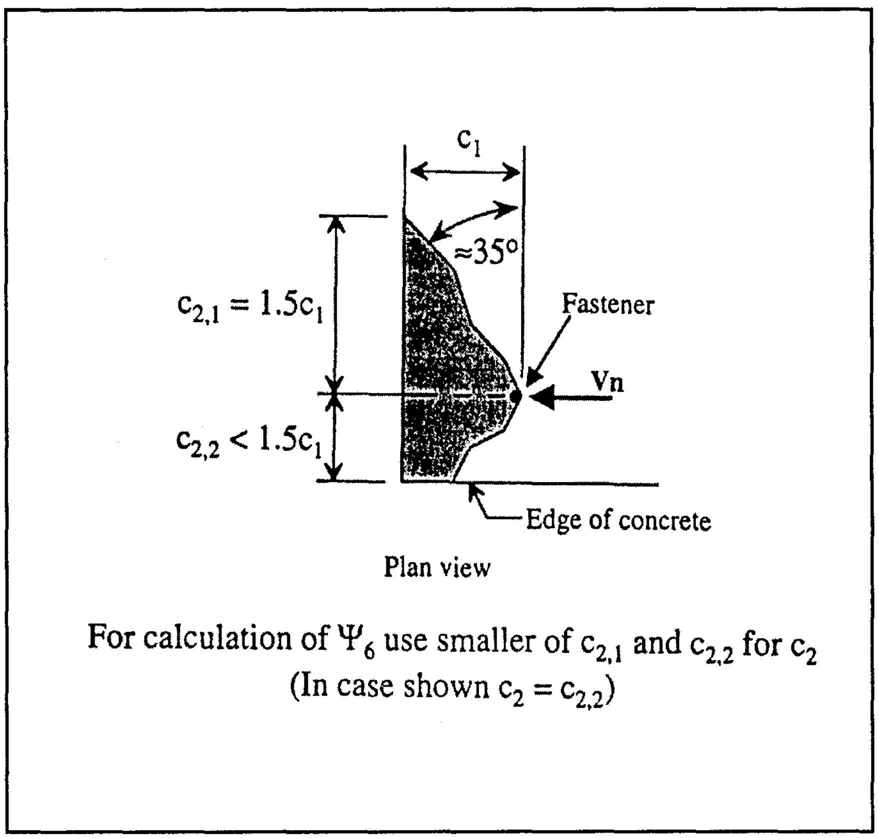


FIGURE C9.2.6.2.6 Dimension c_2 for edge proximity modification factor.

9.2.6.2.7: Torque-controlled and displacement-controlled expansion anchors are permitted in cracked concrete under pure shear loadings.

9.2.6.3 Concrete Pryout Strength:

9.2.6.3.1: The article by Fuchs, Eligehausen, and Breen (1995) indicates that the pryout shear resistance can be approximated as 1 to 2 times the anchor tensile resistance with the lower value appropriate for h_{ef} less than 2.5 in.

9.2.7 Interaction of Tensile and Shear Forces: The shear-tension interaction expression has traditionally been expressed as:

$$\left(\frac{N}{N_n}\right)^{\alpha} + \left(\frac{V}{V_n}\right)^{\alpha} \leq 1.0 \qquad \text{(C9.2.7)}$$

where α varies from 1 to 2. The current tri-linear recommendation is a simplification of the expression where $\alpha = 5/3$ (Figure C9.2.7). The limits were chosen to eliminate the requirement for computation of interaction effects where very small values of the second force are present. However, any other interaction expression that is verified by test data can be used under Sec. 9.2.4.3.

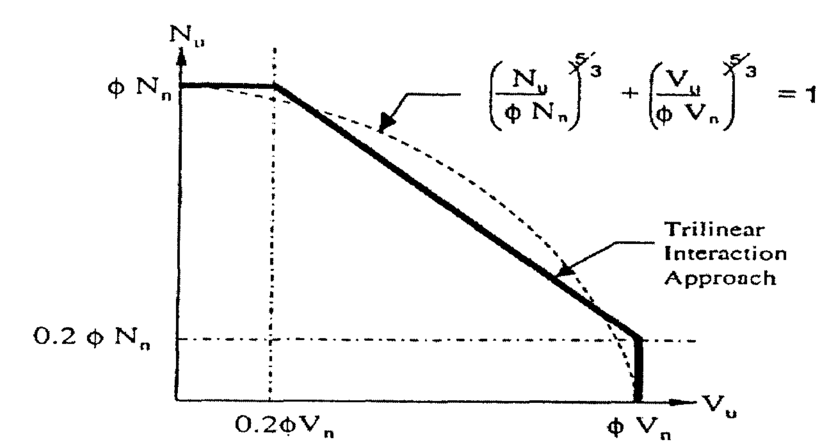


FIGURE C9.2.7 Shear and tensile load interaction equation.

9.2.8 Required Edge Distances, Spacings, and Thicknesses to Preclude Splitting Failure: The minimum spacings, edge distances and thicknesses are very dependent on the anchor characteristics. Installation forces and torques in post-installed anchors can cause splitting of the surrounding concrete. Such splitting also can be produced in subsequent torquing during connection of attachments to anchors including cast-in anchors. The primary source of values for minimum spacings, edge distances, and thicknesses of post-installed anchors should be the product-specific prequalification tests of Sec. 9.2.1.4. However, in some cases specific products are not known in the design stage. Approximate values are provided for use in design.

9.2.8.2: Since the edge cover over a deep embedment close to the edge can have a significant effect on the side-face blowout strength of Sec. 9.2.5.4, in addition to the normal concrete cover requirements, the designer may wish to use larger cover to increase the side-face blowout strength.

9.2.8.3: Drilling holes for post-installed anchors can cause microcracking. The requirement for a minimum edge distance 2 times the maximum aggregate size is to minimize the effects of such microcracking.

9.2.8.5: This minimum thickness requirement is not applicable to through-bolts because they are outside the scope of the Provisions. In addition, splitting failures are caused by the load transfer between the bolt and the concrete. Because through-bolts transfer their load differently than cast-in or expansion and undercut anchors, they would not be subject to the same member thickness requirements. Post-installed anchors should not be embedded deeper than 2/3 of the member thickness.

9.2.9 Installation of Anchors: Many anchor performance characteristics depend on proper installation of the anchor. Anchor capacity and deformations can be assessed by the anchor prequalification tests. These tests are carried out assuming that the manufacturer's installation directions will be followed. Certain types of anchors can be sensitive to variation in hole diameter, cleaning conditions related to embedment depth, orientation of the axis, magnitude of the installation torque, proximity of reinforcement, and other variables. Some of this This sensitivity is indirectly reflected in the assigned ϕ values for the different anchor categories, which depend in part on the results of the installation safety reliability tests. Gross deviations from the prequalification testing results could occur if anchor *components* are incorrectly exchanged or if anchor installation criteria and procedures vary from those recommended. Project specifications should require that anchors be installed according to the manufacturer's recommendations.

9.3 CLASSIFICATION OF SHEAR WALLS: In the 2000 *Provisions*, shearwalls have been classified by the amount and type of detailing required. This classification was developed to facilitate assigning shearwalls to seismic design categories.

9.4 SEISMIC PERFORMANCE CATEGORY A: Construction qualifying under Category A may be built with no special detailing requirements for earthquake resistance. Special details for ductility and *toughness* are not required in Category A.

9.5 SEISMIC PERFORMANCE CATEGORY B: Special details for ductility and *toughness* are not required in Category B.

9.5.1 Ordinary Moment Frames: Since ordinary *frames* are permitted only in Categories A and B, they are not required to meet any particular seismic requirements. Attention should be paid to the often overlooked requirement for *joint* reinforcement in Sec.11.11.2 of ACI 318.

9.6 SEISMIC PERFORMANCE CATEGORY C: A *frame* used as part of the lateral force resisting system in Category C is required to have certain details that are intended to help sustain integrity of the *frame* when subjected to *deformation* reversals into the nonlinear range of response. Such *frames* must have attributes of *intermediate moment frames*. Structural *(shear) walls* of *buildings* in Category C are to be built in accordance with the requirements of ACI 318.

9.6.2 Intermediate Moment Frames and 9.6.3 Special Moment Frames: The concept of *moment frames* for various levels of hazard zones and of performance is changed somewhat from the provisions of ACI 318. Two sets of *moment frame* detailing requirements are defined in ACI 318, one for "regions of high seismic risk" and the other for "regions of moderate seismic risk." For the purposes of this document, the "regions" are made equivalent to Seismic Performance Categories in which "high risk" means Categories D and E and "moderate risk" means Category C. This document labels these two *frames* the "*special moment frame*" and the "*intermediate moment frame*," respectively.

The level of inelastic energy absorption of the two *frames* is not the same. The *Provisions* introduce the concept that the *R* factors for these two *frames* should not be the same. The preliminary version of the Provisions (ATC 3-06) assigned the *R* for ordinary *frames* to what is now called the intermediate *frame*. In spite of the fact that the *R* factor for the intermediate *frame* is less than the *R* factor for the special *frame*, use of the intermediate *frame* is not permitted in the higher Performance Categories (D and E). On the other hand, this arrangement of the provisions encourages consideration of the more stringent detailing practices for the special *frame* in Category C because the reward for use of the higher *R* factor can be weighed against the higher cost of the detailing requirements. The *Provisions* also introduce the concept that an intermediate *frame* may be a part of a Dual System in Category C.

The differences in the performance basis of the requirements for the two types of *frames* might be briefly summarized as follows (see the commentary of ACI 318 for a fuller discussion of the requirement for the special *frame*):

1. The shear *strength* of beams and columns shall not be less than that required when the member has yielded at each end in flexure. For the special *frame*, strain hardening and other factors are considered by raising the effective tensile *strength* of the bars to 125 percent of specified yield. For the intermediate *frame*, an escape clause is provided in that the calculated shear using double the prescribed seismic force may be substituted. Both types require the same minimum amount and maximum spacing of transverse reinforcement throughout the member.

2. The shear *strength* of *joints* is limited and special provisions for anchoring bars in *joints* exist for *special moment frames* but not intermediate *frames*. Both *frames* require transverse reinforcement in *joints* although less is required for the intermediate *frame*.

3. Closely spaced transverse reinforcement is required in regions of potential hinging (typically the ends of beams and columns) to control lateral buckling of longitudinal bars after the cover has spalled. The spacing limit is slightly more stringent for columns in the special *frame*.

4. The amount of transverse reinforcement in regions of hinging for special *frames* is empirically tied to the concept of providing enough confinement of the concrete core to preserve a ductile response. These amounts are not required in the intermediate *frame* and, in fact, stirrups in lieu of hoops may be used in beams.

5. The special *frame* must follow the strong column/weak beam rule. Although this is not required for the intermediate *frame*, it is highly recommended for multistory construction.

6. The maximum and minimum amounts of reinforcement are limited to prevent rebar congestion and assure a nonbrittle flexural response. Although the precise limits are different for the two types of *frames*, a great portion of practical, buildable designs will satisfy either.

7. Minimum amounts of continuous reinforcement to account for moment reversals are required by placing lower limits on the flexural *strength* at any cross section. Requirements for the two types of *frames* are similar.

8. Locations for splices of reinforcement are more tightly controlled for the special *frame*.

9. In *addition*, the special *frame* must satisfy numerous other requirements beyond the intermediate *frame* to assure that member proportions are within the scope of the present research experience on seismic resistance and that the analysis, the design procedures, the qualities of the materials, and the inspection procedures are at the highest level of the state of the art.

9.7 SEISMIC PERFORMANCE CATEGORIES D, E, or F: The requirements conform to current practice in the areas of highest seismic hazard.

REFERENCES

ACI Committee 349, “Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-85), Appendix B. - Steel Embedement,” ACI Manual of Concrete Practice, Part 4, 1987.

ACI Committee 550. 1993. "Design Recommendations for Precast Concrete Structures" *ACI Structural Journal* 90 (1):115-121.

ACI Innovation Task Group 1 and Collaborators. 1999. "Acceptance Criteria for Moment Frames Based on Structural Testing," American Concrete Institute, Farmington Hills, MI.

ANSI/ASME B1.1, “Unified Inch Screw Threads” (UN and UNR Thread Form), ASME, Farifield, NJ, 1989.

ANSI/ASME B18.2.1, “Square and Hex bolt and Screws, Inch series,” ASME, Fairfield, NJ, 1996.

ANSI/ASME B18.2.6., “Fasteners for Use in Structural Applications,” ASME, Fairfield, NJ 1996.

Applied Technology Council. 1981. *Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads* , Report ATC-8.

Building Seismic Safety Council. 1987. *Guide to Use of NEHRP Recommended Provisions in Earthquake Resistant Design of Buildings,* 1985 Edition. Washington, D.C.: FEMA.

Cheok, G. S., and H. S. Lew. 1991. "Performance of Precast Concrete Beam-to-Column Connections Subject to Cyclic Loading," *PCI Journal*, 36, (3): 56-67.

Cheok, G.S., W.C. Stone, and S.K. Kunnath. 1998. "Seismic Response of Precast Concrete Frames with Hybrid Connections," *ACI Structural Journal* 95(5):527-539.

Clough, D.P. 1986. " A Seismic Design Methodology for Medium-Rise Precast Concrete Buildings." In *Proceedings of Seminar on Precast Concrete Construction in Seismic Zones,* JSPS/NSF, Tokyo, October 1986.

Cook, R.A. and Klingner, R.E., "Behavior of Ductile Multiple-Anchor Steel-to-COncrete connections with Surface Mounted Baseplates," Anchors in Concrete: Design and Behavior, American Concrete Institute Special Publication SP-130, February 1992, pp. 61-122.

Cook, R.A. and Klingner, R.E., "Ductile Multiple-Anchor Steel-to-Concrete Connections," Journal of Structural Engineering, ASCE, Vol. 118, No 6, June 1992, pp. 1645-1665.

Cook, R.A. 1999. "Strength Design of Anchorage to Concrete", Portland Cement Association, Skokie, IL, 1-13.

Design of Fastenings in Concrete, Comite Euro International du Beton (CEB), Thomas Telford Services Ltd., London, Jan. 1997.

Eligehausen, R., and Balogh, T., "Behavior of Fasteners Loaded in Tension in Cracked Reinforced Concrete," ACI Structural Journal, Vol. 92, No. 3, May-June 1995, pp. 365-379.

Eligehausen, R., Fuchs, W., and Mayer, B., "Load Bearing Behavior of Anchor Fastenings in Tension," Bentonwerk + Fertigteiltechnik, 12/1987, pp. 826-832, and 1/88, pp. 29-35.

Eligehausen, R., and Fuchs, W., "Load Bearing Behavior of Anchor Fastenings under Shear, Combined Tension and Shear orFlexural Loadings," Bentonwerk + Fertigteiltechnik, 2/1988, pp.48-56.

Elliott, K. S., G. Davies, and W. Omar. 1992. "Experimental and Theoretical Investigation of Precast Concrete Hollow-Cored Slabs Used as Horizontal Floor Diaphragms," *The Structural Engineer*, 70, (10):175-187.

El-Sheikh, M.T., R. Sause, S. Pessiki, and L.W. Lu. 1999. "Seismic Behavior and Design of Unbonded Post-tensioned Precast Concrete Frames." *PCI Journal* 44(3):54-69.

Englekirk, R. E. 1987. "Concepts for the Development of Earthquake Resistant Ductile Frames of Precast Concrete," *PCI Journal* , 32,(1).

Farrow, C.B., and Klingner, R.E., "Tensile Capacity of Anchors with Partial or Overalpping Failure Surfaces: Evaluation of Existing Formulas on an LRFD Basis," ACI Structural Journal, Vol. 92, No. 6, Nov.-Dec. 1995, pp. 698-710.

Fastenings to Concrete and Masonry Structures, State of the Art Report, Comite Euro-International du Beton (CEB), Bulletin No. 216, Thomas Telford Services Ltd., London, 1994.

French, C. W., M. Hafner, and V. Jayashanker. 1989. "Connections Between Precast Elements - Failure Within Connection Region," *ASCE Journal of Structural Engineering*, 115, (12):3171-3192.

Fuchs W., Elighausen, R., and Breen, J., "Concrete Capacity Design (CCD) Approach for Fastening to Concrete," ACI Structural Journal, Vol. 9, No.1, Jan.-Feb. 1995, pp.73-93. Discussion - ACI Structural Journal, Vol. 92, No. 6, Nov.-Dec. 1995, pp. 787-82.

Furche, J., and Eligehausen, R., "Lateral Blow-out Failure ofHeaded Studs Near a Free Edge," Anchors in Concrete - Design and Behavior, SP 130, ACI, Detroit, 1991, pp. 235-252.

Ghosh, S.K., S.D. Nakaki, and K. Krishnan. 1997. "Precast Structures in Regions of High Seismicity: 1997 UBC Design Provisions," *PCI Journal* 42(6): 76-91.

Hawkins, N. M., and R. E. Englekirk. 1987. "U.S.-Japan Seminar on P/C Concrete Construction in Seismic Zones," *PCI Journal*, 32,(2).

Jayashanker, V., and C. E. French. 1988. "An Interior Moment Resistant Connection Between Precast Elements Subjected to Cyclic Lateral Loads," *Structural Engineering Report No. 87-10*, Minneapolis: University of Minnesota.

Klingner, R., Mendonca, J., and Malik, J., "Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts under Reversed Cyclic Loading," Journal of the American Concrete Institute, Vol. 79, No. 1, 1982, pp. 3-12.

Kurama, Y., S. Pessiki, R. Sause, and L.W.Lu. 1999. "Seismic Behavior and Design of Unbonded Post-tensioned Precast Concrete Walls." *PCI Journal* 44(3): 72-89.

Kuhn, D., and Shaikh, F., "Slip-Pullout Strength of Hooked Anchors," Research Report, University of Wisconsin - Milwaukee, 1996.

Lotze, D. and Klingner, R.E., "Behavior of Multiple-Anchor Attachments to Concrete from the Perspective of Plastic Theory," Report PMFSEL 96-4, Ferguson Structural Engineering Laboratory, The University of Texas at Austin, March 1997.

Lutz, L., "Discussion to Concrete Capacity Design (CCD) Approach for Fastening to Concrete," ACI Structural Journal, Nov./Dec. 1995, pp. 791-792 and authors closure pp. 798-799.

Mast, R. F. 1992. "A Precast Concrete Frame System for Seismic Zone Four," *PCI Journal* 37(1):50-64.

Menegotto, M. 1994. " Seismic Diaphragm Behavior of Untopped Hollow-Core Floors." In Proceedings, Federation International of Prestressing (FIP) Congress, Washington, D.C., May.

Mueller, P. 19896. "Hysteretic Behavior of Precast panel Walls." In Proceedings, Seminar on precast Concrete Construction in Seismic Zones, JSPS/NSF, Tokyo, 1989, Vol.1, pp. 127-142.

Nakaki, S.D. 2000. "Design Guidelines for Precast and Cast-in-Place Concrete Diaphragms," 1998 NEHRP Professional Fellowship Report, Earthquake Engineering Research Institute, Oakland, CA.

Nakaki, S. D., and R. E. Englekirk. 1991. "PRESSS Industry Seismic Workshops: Concept Development," *PCI Journal* 36(5):54-61.

Nakaki, S.D., Stanton, J.F. and Sritharan, S.. 1999 "An Overview of the PRESSS Five Story Precast Test Building," PCI Journal 44 (2): 26-39.

Neille, D. S. 1977. "Behavior of Headed Stud Connections for Precast Concrete Connections for Precast Concrete Panels Under Monotonic and Cycled Shear Loading," thesis submitted in partial fulfillment of the requirements of Doctor of Philosophy, University of British Columbia.

New Zealand Society for Earthquake Engineering. 1991. "Guidelines for the Use of Structural Precast Concrete in Buildings,".

Pekau, O. A., and D. Hum. 1991. "Seismic Response of Friction-Jointed Precast Panel Shear Walls," *PCI Journal* 36(2):56-71.

PCI Design Handbook, 4th Edition, Precast/Prestressed Concrete Institute, Chicago 1992, pp. 570.

Pincheira, J.S., M.G. Oliva, and F.I. Kusumo-Rahardjo. 1998. "Tets on Double Tee Flange Connectors Subjected to Monotonic and Cyclic Loading," *PCI Journal* 43(4):82-96.

Powell, G., F. Filippou, V. Prakash, and S. Campbell. 1993. "Analytical Platform for Precast Structural Systems," *In Proceedings, ASCE Structures Congress '93*. New York: ASCE.

Priestley, M. J. N. 1991. "Overview of PRESSS Research Program," *PCI Journal* 36(4):50-57.

Priestley, M.J.N. 1996. "The PRESSS Program-Current Status and Proposed Plans for Phase III," *PCI Journal* 41(2):22-40.

Priestley, M.J.N.,S. Sritharan, J.R. Conley, and S. Pampanin. 1999. "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building," *PCI Journal* 44(6).

Priestley, M. J. N., and J. T. Tao. 1993. "Seismic Response of Precast Prestressed Concrete Frames with Partially Debonded Tendons," *PCI Journal* 38(1):58-69.

Primavera, E.J., Pinelli, J.P. and Kalajian, E.H., "Tensile Behavior of Cast-in-Place and Undercut Anchors in High-Strength Concrete," ACI Structural Journal, Vol. 94, No. 5, Sept-Oct. 1997, pp. 583-594.

Schultz, A.E. and R.A. Magana. 1996. "Sesimic Behavior of Connections in Precast Concrete Walls," Mete A. Sozen Symposium , American Concrete Institute, Farmington Hills, MI, pp: 273-311.

Seo, S.-Y., L.-H. Lee, and N. M. Hawkins. 1998. "The Limiting Drift and Energy Dissipation Ratio for Shear Walls Based on Structural Testing," Journal of the Korean Concrete Institute 10(6):335-343.

Shaikh, A.F., and Yi, W., "In-place Strength and Welded Studs," PCI Journal, Vol 30(2), March-Apr. 1985.

Stanton, J. F., R. G. Anderson, C. W. Dolan, and D. E. McClearly. 1986. *"Moment-Resisting and Simple Connections,"* Research Report 1/4, Prestressed Concrete Institute, 1986.

Stanton, J. F., T. R. Hicks, and N. M. Hawkins. 1991. "PRESS Project 1.3: Connection Classification and Evaluation," *PCI Journal* 36(5):62-71.

Stanton, J., W.C.Stone, and G.S. Cheok. 1997. "A Hybrid Reinforced Precast Frame for Seismic Regions,"*PCI Journal* 42(2):20-32.

Warnes, C. E. 1992. "Precast Concrete Connection Details for All Seismic Zones," *Concrete International* 14(11): 36-44.

Wong, T.L., "Stud Groups Loaded in Shear" M.S. Thesis, Oklahoma State University, 1988.

Wood, S. L. 1980. "Shear Strength of Low-Rise Reinforced Concrete Walls," ACI Structural Journal 87(1): 99-107.

Yee, A. A. 199. "Design Considerations for Precast Prestressed Concrete Building Structures in Seismic Areas," *PCI Journal* 36(3):40-55.

Zhang, Y., "Dynamic Behavior of Multiple Anchor Connections in Cracked Concrete," Ph.D. Dissertation, The University of Texas at Austin, August1997.

Prestressed and Partially Prestressed Concrete

ACI-ASCE Committee 423. 1985. "Recommendations for Concrete Members Prestressed with Unbonded Tendons," *ACI Manual for Concrete Practice*," Part 3, p. 423.3-1 - 423.3-R16, American Concrete Institute, Detroit, Michigan.

Ishizuka, T., and N. M. Hawkins. 1987. "Effect of Bond Deterioration on the Seismic Response of Reinforced and Partially Prestressed Concrete and Ductile Moment Resistant Frames," SM87-2, University of Washington, Department of Civil Engineering, Seattle, Washington.

JSPS/NSF. 1986. *Proceedings of the Seminar on Precast Construction in Seismic Zones,* Vol. 1.

Park, R., and K. J. Thomspon.1977. "Cyclic Load Tests on Prestressed and Partially Prestressed Beam-Column Joints," *PCI Journal* 22(5):84-110.

Thompson, K. J., and R. Park. 1980. "Seismic Response of Partially Prestressed Concrete," *ASCE Journal of the Structural Division* 106(ST8):1755-1775.

Appendix to Chapter 9

REINFORCED CONCRETE DIAPHRAGMS CONSTRUCTED USING UNTOPPED PRECAST CONCRETE ELEMENTS

C9A.1 BACKGROUND: Although not directly addressed in the code, untopped precast *components* have been used as diaphragms in high seismic regions. Untopped hollow core with grouted joints and end chords have performed successfully both in earthquakes and in laboratory tests, (Elliot et al., 1992) (Menegotto,1994), (Priestley et al. 1999). Experience has also demonstrated the unsuccessful use of cast-in-place concrete topping as diaphragms (Iverson and Hawkins, 1994). Where problems have occurred, they have not been inherently with the precast construction, but the result of a failure to address fundamental requirements of structural mechanics.

This section provides conditions that are intended to ensure that diaphragms composed of precast *components* are designed with attention to the principles required for satisfactory behavior. Each condition addresses requirements that should be considered for all diaphragms, but which are particularly important in jointed construction. Specific attention should be paid to providing a complete load path that considers force transfer across all joints and connections.

C9A.3 Untopped Precast Diaphragms:

C9A.3.1: Out-of-plane offsets in the vertical elements of the seismic-force-resisting system place particularly high demands on the diaphragm in providing a continuous load path. Untopped precast diaphragms are not suitable for this condition.

C9A.3.2: Following the principle that the diaphragm is not generally an appropriate location for inelastic behavior and, in particular, for untopped precast diaphragms, specific direction is provided that elastic models should be used for diaphragm analysis. Connections are subject to a combination of load effects (Fleischman et al. 1998). The distribution of loads may change after yielding, and therefore the design of the diaphragm should avoid yielding.

C9A.3.3: Since the diaphragm is not generally an appropriate location for inelastic behavior, it should be designed to a level of strength that is intended to ensure the ductility and yield strength of the seismic-force-resisting system can be mobilized before the diaphragm yields. While research (Fleischman et al. 1998) suggests that the diaphragm demand will not exceed twice the equivalent lateral forces used for the vertical system design, Table 5.2.2. prescribes an overstrength factor, Ω_o, and Sec. 5.2.4 prescribes a redundancy factor, ρ, for the systems that should be used. If an analysis of the probable strength of the seismic-force-resisting system is made to determine a lower demand on the diaphragm, the design force used should still be sufficient to attempt to ensure that the diaphragm remains elastic. For that reason a 1.25 factor is specified.

C9A.3.4: It must be recognized that the demand on diaphragms in buildings with these plan irregularities requires special attention. In accordance with Sec 5.2.6.4.3 the design force for the diaphragm should be increased by at least 25 percent when such irregularities are present in structures assigned to SDC D, E and F.

C9A.3.5: Although the design procedures prescribed in these sections are intended to ensure elastic behavior at the level of the code design forces, it is recognized that catastrophic events may exceed code requirements. Under such circumstances, it is important that the connections possess ductility under reversed cyclic loading. The intent, in these sections, is for the connection capacity to be limited by steel yielding of the connector and not by brittle concrete failure or weld fracture.

C9A.3.6: Substantiating experimental evidence to demonstrate through testing and evaluation that mechanical connections satisfy the principles specified in ITG/T1.1 and ATC-24, and can develop the required capacity and ductility, should meet the following criteria:

Test Procedures:

1. Prior to testing, a design procedure should have been developed for prototype connections having the generic form that is to be tested for acceptance.
2. That design procedure should be used to proportion the test specimens.
3. Specimens should not be less than two-thirds scale.
4. Test specimens should be subject to a sequence of reversing cycles having increasing limiting displacements.
5. Three fully reversed cycles should be applied at each limiting displacement.
6. The maximum load for the first sequence of three cycles should be 75 percent of the calculated nominal strength of the connection, E_n.
7. The stiffness of the connection should be defined as 75 percent of the calculated nominal strength of the connection divided by the corresponding measured displacement, δ_m.
8. Subsequent to the first sequence of three cycles, limiting displacements should be incremented by values not less than one, and not more than one and one quarter times δ_m.

Acceptance Criteria:

1. The connection should develop a strength, E_{max}, greater than its calculated nominal strength, E_n..
2. The strength, E_{max}, should be developed at a displacement not greater than $3\delta_m$.
3. For cycling between limiting displacements not less than $3\delta_m$, the peak force for the third loading cycle for a given loading direction should not be less than $0.8\ E_{max}$ for the same loading direction.

Results of reversed cyclic loading tests on typical connections are reported in Spencer (1986) and Pincheira et al. (1998).

C9A.3.9: Successful designs may include a combination of untopped precast *components* with areas of concrete topping in locations of high force demand or concentration. Such topping can allow for continuity of reinforcement across joints. For such designs, the requirements for topping slab diaphragms apply to the topped portions.

C9A.3.10: .An important element in the *Provisions* is attention to deformation compatibility requirements. Reduction in effective shear and flexural stiffness for the diaphragm is appropriate in evaluating the overall effects of drift on elements that are not part of the seismic-force-resisting system. This approach should encourage the use of more vertical elements to achieve shorter spans in the diaphragm and result in improved system redundancy and diaphragm continuity. Redundancy will also improve the overall behavior should any part of the diaphragm yield in a catastrophic event.

REFERENCES:

Elliot, K.S., G. Davies, and W. Omar, "Experimental and Theoretical Investigation of Precast Concrete Hollow_cored Salbs Used as Horizontal Floor Diaphragms." The Structural Engineer, V. 70, No. 10, May 1992, pp.175-187.

Fleischman, R.B., Sause, R., Pessiki, S., and A.B. Rhodes, "Seismic Behavior of Precast Parking Structure Diaphragms," PCI Journal, V. 43, No. 1, Jan.-Feb. 1998, pp. 38-53.

Iverson, J.K., and N.M. Hawkins, "Performance of Precast/Prestressed Concrete Building Structures During the Northridge Earthquake," PCI Journal, V. 39, No. 2, March-April 1994, pp. 38-55.

Menegotto, M. 1994. "Seismic Diaphragm Behavior of Untopped Hollow-Core-Floors," Proceedings, F.I.P. Congress, Washington, D.C.

Pincheira, J.A., Oliva, M.G., and F.I. Kusumo-Rahardjo, "Tests on Double Tee Flange Connectors Subject to Monotonic and Cyclic Loading, " PCI Journal, V. 43, No. 3, May-June 1998, pp. 82-96.

Priestley, M.J.N., Sritharan, S., Conley, J.R., and S. Pampanin, "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building, PCI Journal, V. 44, No. 6, Nov.-Dec. 1999.

Spencer, R., "Earthquake Resistant Connections for Low Rise Precast Concrete Buildings," Proceedings, U.S.-Japan Seminar on Precast Concrete Construction in Seismic Zones,V. 1, Japan Concrete Institute, October 1986.

Chapter 10 Commentary

COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS

The 1994 Edition of the *NEHRP Recommended Provisions* included a new chapter on composite steel and concrete structures. The Provisions have been updated and incorporated in Part II of the 1997 Edition of the AISC *Seismic Provisions*. This edition of the *NEHRP Recommended Provisions* includes by reference Part II of the AISC *Seismic Provisions (1997),* together with the underlying AISC-LRFD (1999) and ACI-318 (1999) standards. Part II of the AISC *Seismic Provisions* provides definitions for composite systems consistent with the system designations in Table 5.2.2 and specifies requirements for the seismic design of composite systems and components.

In general, available research shows that properly detailed composite elements and connections can perform as well, or better, than structural steel and reinforced concrete *components*. However, due to the lack of design experience with certain types of composite *structures* in high seismic risk areas, usage of composite systems in *Seismic Design Categories* D and above requires documentation (substantiating evidence) that the proposed system will perform as intended by Part II of the AISC *Seismic Provisions* and implied by the R values in Table 5.2.2. It is intended that the substantiating evidence consist of a rational analysis that considers force transfer between structural steel, reinforced concrete and composite elements and identifies locations in the *structure* required to sustain inelastic deformations and dissipate seismic energy. Design of composite members and connections to sustain inelastic deformations shall be based on models and criteria substantiated by test data. For many composite *components*, test data and design models are available and referenced in the commentary to the AISC *Seismic Provisions* – Part II (1997).

Chapter 11 Commentary

MASONRY STRUCTURE DESIGN REQUIREMENTS

11.1 GENERAL:

11.1.1 Scope: The provisions of Chapter 11 govern design and construction of all types of masonry. Quality assurance is covered with a reference to Chapter 3. Reinforced and plain (unreinforced) masonry elements that are part of the basic structural system and those that are not part of the basic structural system are included.

11.1.2 Reference Documents: Design and construction standards cited in Chapter 11 are listed in Sec. 11.1.2. The materials standards are specifically listed to include only those materials permitted by the provisions. The listing includes the document's designation, the year of the edition and the title of the document.

11.1.3 Definitions: Terms used in the provisions which have a specific meaning which differs from the dictionary definition are defined in Sec. 11.1.3. All other terms are defined by the dictionary.

11.1.4 Notations: Notations used in the provisions are defined in Sec. 11.1.4. English units of measure are stated followed by the metric unit in parenthesis for each term.

11.2 CONSTRUCTION REQUIREMENTS:

11.2.1 General: ACI 530.1 is a standard specification prepared under consensus procedures. It was developed by members representing construction, design, materials, and research of masonry structures. The document is intended to be incorporated into contract documents used to construct masonry structures.

This standard specification was developed to be used in conjunction with *Building Code Requirements for Masonry structures*, ACI 530. Appropriate standards for materials and test methods are referenced. In addition to a general section, there are sections on masonry, reinforcement and metal accessories, and grout.

The materials listed in ACI 530.1 have been restricted in order to obtain more predictable behavior and better performance required for strength design. Construction provisions found in Chapter 11 override those found in ACI 530.1.

11.2.2 Quality Assurance: See Chapter 3 of the *Provisions* and *Commentary*. Quality assurance requirements for masonry structures include testing of masonry *components* (mortar, grout, and units) or testing of masonry assemblages. Industry guidelines for materials testing are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

No. 39 Revised, "Testing for Engineered Brick Masonry: Brick, Mortar and Grout," January 1987.

No. 39A, "Testing for Engineered Brick Masonry: Determination of Allowable Design Stresses," December 1987.

No. 39B, "Testing for Engineering Brick Masonry: Quality Assurance," March 1988.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

TEK 22A, *Prism Testing for Engineered Concrete Masonry*, 1979.

TEK 107, *Laboratory and Field Testing of Mortar and Grout*, 1979.

TEK 108, *Testing Concrete Masonry Assemblages*, 1979.

Industry guidelines for field inspection are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

No. 17C, "Reinforced Brick Masonry: Inspectors' Guide," May 1986.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

TEK 65, *Field Inspection of Engineered Concrete Masonry*, 1975.

TEK 132, *Inspector's Guide for Concrete Masonry Construction*, 1983.

11.3 GENERAL REQUIREMENTS:

11.3.1 Scope: This chapter offers three different methods for designing masonry structures. Any method, used within the limitations imposed, provides acceptable masonry construction with acceptable seismic resistance characteristics.

11.3.2 Empirical Masonry Design: Empirical design methods are based on the successful performance of masonry buildings. Prescriptive requirements and limited exposure to loads are necessary to ensure compliance.

The design process results in sizes and proportions of masonry elements using minimum thicknesses and maximum spans. Although rudimentary stress calculations are made, empirical masonry design does not require a complete structural analysis.

11.3.3 Plain (Unreinforced) Masonry Design: Design methods for plain masonry, often referred to as unreinforced masonry. The procedures utilize working stress design requirements using principles of mechanics.

11.3.4 Reinforced Masonry Design: Reinforcing steel complements the high compressive strength of masonry with high tensile strength. Increased load-carrying capacity and greater ductility result from the use of reinforcing steel.

11.3.5 - 11.3.9 Seismic Design Categories A through F: Any type of masonry shear wall is permitted in Seismic Design Categories A and B. Detailed plain masonry shear walls or intermediate reinforced masonry shear walls are required for Seismic Design Category C. Special reinforced masonry shear walls are required for Seismic Design Categories D, E, or F. Minimum requirements for each type of masonry shear wall are given in Sec. 11.11. These requirements are consistent with intended inelastic deformation capacities that are the bases for the R, Ω, and C_d factors given in Table 5.2.2. Additional requirements for construction of masonry elements other than shear walls are given for each Seismic Design Category in Sec. 11.3.5 through 11.3.9

11.3.6 Seismic Design Category B: The use of empirical masonry design, Sec. 11.3.2, for the lateral load resisting system is not appropriate for Seismic Design Category B. Masonry walls that are not part of the lateral load resisting system may be designed by the empirical method.

11.3.10: Properties of Materials:

11.3.10.1 Steel Reinforcement Modulus of Elasticity: The given modulus of elasticity of steel reinforcement is taken from previous codes and is consistent with established design values. Design may be based on tested values of modulus of elasticity; however, these tests are rarely performed because it is impractical to test materials to be used in the construction at the time when the project is being designed.

11.3.10.2 Masonry Modulus of Elasticity: Modulus of elasticity of masonry is used in determining stiffness of structural *components* prior to cracking. Therefore, the modulus is taken from the elastic portion of the stress strain curve. The modulus of elasticity of masonry is not clearly related to any property of mortar, unit, grout or prism *h/t*, but is influenced by all of these. TS5 concluded it was best to relate the value of E_m to the specified compressive strength of masonry. This is because f_m' is also influenced by these parameters. The 750 multiplier is used rather than lower multipliers reported (Wolde-Tinsae, 1993) since the actual compressive strength of masonry must exceed the specified compressive strength.

11.3.10.4 Masonry Compressive Strength: Research has been performed on structural masonry *components* having a compressive strength in the range of 1,500 to 6,000 psi (10 to 41 MPa). Design criteria are based on these research results. Design values therefore are limited to compressive strengths in the range of 1,500 to 4,000 psi (10 to 28 MPa) for concrete masonry and 1,500 to 6,000 psi (10 to 41 MPa) for clay masonry.

11.3.10.5 Modulus of Rupture: Modulus of rupture values in Table 11.3.10 are based on allowable working stress values for flexural tension multiplied by 2.0 to approximate the lower limit of strength values. See the Commentary to ACI 530 for discussion. Stack bond masonry has historically been assumed to have no flexural bond strength across the head joints; thus, the grout area alone is used.

11.3.10.6 Reinforcement Strength: Research conducted on reinforced masonry *components* used Grade 60 reinforcement. To be consistent with laboratory documented performance, design is based on a steel yield strength that does not exceed 60,000 psi (413 MPa).

11.3.11 Section Properties: Section properties of masonry members are available in masonry design publications. Design is based on specified dimension. Actual dimensions may vary within the tolerance range given in the construction requirement (i.e., ACI 530.1). The strength reduction factors are based in part on an anticipated variation in the specified (design) dimensions.

11.3.12 Headed and Bent-Bar Anchor Bolts: This section covers cast-in-place headed anchor bolts and bent-bar anchors (J- or L-bolts) in grout. General background information on this topic is given in CEB, 1995.

The tensile capacity of a headed anchor bolt is governed by yield and fracture of the anchor steel or by breakout of a roughly conical volume of masonry starting at the anchor head and having a fracture surface oriented at 45 degrees to the masonry surface. Steel capacity is calculated conventionally using the effective tensile stress area of the anchor (i.e., including the reduction in area of the anchor shank due to threads). Masonry breakout capacity is calculated using expressions adapted from concrete design, which use a simplified design model based on a stress of $4\sqrt{f_m'}$ uniformly distributed over the area of that right circular cone, projected onto the surface of the masonry. Reductions in breakout capacity due to nearby edges or adjacent anchors are computed in terms of reductions in those projected areas (Brown and Whitlock, 1983).

The tensile capacity of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile cone breakout of the masonry, or by straightening and pullout of the anchor from the masonry. Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. Pullout capacity is calculated as proposed by Shaikh, 1996. Possible contributions to tensile pullout capacity due to friction are neglected.

The tensile breakout capacity of a headed anchor is usually much greater than the pullout capacity of a J- or L-bolt. the designer is encouraged to use headed anchors when anchor tensile capacity is critical.

The shear capacity of a headed or a bent-bar anchor bolt is governed by yield and fracture of the anchor steel or by masonry shear breakout. Steel capacity is calculated conventionally using the effective tensile stress area (i.e., threads are conservatively assumed to lie in the critical shear plane). Shear breakout capacity is calculated as proposed ;by Brown and Whitlock, 1983.

Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear, however, available research suggests that straightening and pullout may occur. Headed anchor bolts are recommended for such applications (Malik et al., 1982).

11.4 DETAILS OF REINFORCEMENT:

11.4.5 Development of Reinforcement:

11.4.5.2 Development of Reinforcing Bars & Wires in Tension: In order to have ductile behavior of a masonry member subjected to seismic loads, the strength of the bar or wire must be developed by embedment. The development length given by Eq. 11.4.5.2 is based on an analysis of the results of multiple independent research efforts (NCMA, U.S. Army Corps of Engineers by Atkinson-Noven and Associates, and Washington State University) investigating the performance of lap splices and the requirements for development of reinforcement.

Using the compiled data from these studies, numerous multiple regression analyses were performed to identify the parameters having a significant effect on lap splice and development length. The most important parameters are compressive strength of the masonry assemblage (prism strength), diameter of the reinforcing bar, spacing of the bars, and cover. The best-fit equation developed in the regression analyses was simplified for design purposes to Eq. 11.4.5.2 while retaining all the essential parameters.

The lap lengths required by the proposed equation provide a capacity in excess of 1.25 f_y. (Note that an additional factor to account for material variability, construction defects, and other design uncertainties is included by use of the phi factor in the strength design formula.)

11.5 STRENGTH AND DEFORMATION REQUIREMENTS:

11.5.3 Design Strength: The design strength of a member and its connections is calculated by engineering principles and materials strength and yield values. This calculated strength is the nominal strength of the member. The nominal strength is less than the expected or mean strength because minimum guaranteed values or specified strengths are used for the calculations of nominal strength. A strength reduction factor, ϕ, is used to reduce the nominal strength to a design strength. The strength reduction factor, ϕ, is a variable that is dependent on the material and material behavior. Flexural strength of reinforced members is reduced less by the ϕ factor than is shear strength. Exceeding of the flexural strength of a reinforced member causes yielding of the reinforcement but not strength degradation. Exceeding of the shear strength results in a strength degradation.

Flexure Without Axial Load: The strength reduction factor for reinforced masonry is greater than for plain masonry because plain masonry after cracking lacks ductile performance.

Axial Load and Axial Load with Flexure: If the axial load results in balanced strain conditions (flexure produces strain in the reinforcement equal to the yield strain and strain in the masonry equal to the maximum usable strain, ϵ_{mu}) and the flexural reinforcement is minimal, an increase in flexural moment can cause compressive stresses in excess of the compressive strength. The failure will not be ductile; therefore, the strength reduction factor is more severe. Linear interpolation of the strength reduction factor is allowed since the required axial strength due to factored load, P_u, decreases from the axial load resulting in balanced strain conditions to zero, so as to make the transition linear from axial load with flexure to flexure without axial load.

The strength reduction factor for the vertical members of wall frames is more restrictive than for shear walls or coupled shear walls. The strength reduction factor for the vertical members of wall frames does not have a linear variation to its value. When $P_u/A_n f'_m$ is equal to 0.1, the strength reduction factor will be equal to 0.65.

The strength reduction factor for plain masonry members is unchanged from that factor that is applied for flexure only. Axial load increases the flexural capacity of plain masonry but does not significantly change its lack of ductility.

Shear: Strength reduction factors for calculation of design shear strength are commonly more severe than those factors used for calculation of design flexural strength. This concept is partially supported by the wider variance of shear capacities that have been obtained from experimental testing. The variance of the results of each experiment from the body of data is due not only to the variability of the masonry materials, the test apparatus and test methods, and the shear strength parameters tested but also to the greater sensitivity of shear resistance mechanisms to those factors.

Bearing: Exceeding of the bearing capacity causes crushing and spalling of bearing surfaces. The strength reduction factors given are those established for elements that have strength degradation.

11.5.4 Deformation Requirements: Stiffness of a structural element is as important or more important than strength. Stiffness is critical for serviceability and control of displacements. Drift of an element is the movement of one story of the building relative to the adjacent stories or the displacement of the shear wall relative to its fixed base. Drift of the top level of a shear wall is affected by foundation flexibility but the structural stresses and strains in the wall would not be increased by foundation flexibility.

The product of the effective moment of inertia, I, and the effective modulus of elasticity, E, is usually used as a variable for the calculation of the deformation of reinforced elements. The variability in I is caused by tensile cracking of the masonry cross section. If tensile cracking is not acceptable, as for plain masonry, I has a single value and the compressive modulus of elasticity and the moment of inertia of the gross cross section is used for the calculation of deformation.

If tensile cracking in anticipated, such as for reinforced masonry, the effective I at every cross section of the wall or beam is dependent on the curvature of the cross section and the shear deformation of each increment of the member length. Several nonlinear finite element programs have the capability of determining the stiffness degradation of reinforced masonry elements, but the effective stiffness, I, can be determined by use of Eq. 11.5.4.3.

The cracking moment is calculated using the section modulus of the gross section of wall times the modulus of rupture of masonry, f_r. The moment of inertia of the cracked section is calculated about the neutral axis of the section, using the masonry properties, and transforming the reinforcement into equivalent masonry areas by use of the ratio of the compressive modulus of steel and masonry. The cracked moment of inertia, I_{cr}, and the compressive modulus of masonry, E_m, is used to calculate the effective moment of inertia, I_{eff}.

Eq. 11.5.4.3 has been used as a means of providing a transition in stiffness between gross moment of inertia and a totally cracked section. Abboud (1987), Abboud and Hamid (1987), Abboud et al. (1990 and 1993), Hamid et al. (1989), and Horton and Tadros (1990) give additional insight and behavior for computing deflection for masonry *components*.

11.6 FLEXURE AND AXIAL LOADS:

11.6.2 Design Requirements of Reinforced Masonry Members: The design principles listed are those that traditionally have been used for reinforced masonry members. The theory used for design of normally proportioned flexural members has limited applicability to deep flexural members. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stresses in the body of the deep flexural members require that deep beam theory be used for members that exceed the specified limits of span to depth ratio.

11.6.2.2: Longitudinal reinforcement in flexural members is limited to a maximum amount to ensure that masonry compressive strains will not exceed ultimate values: -- in other words, that the compressive zone of the member will not crush before the tensile reinforcement develops the inelastic strain consistent with the maximum drift limits of Provisions Table 5.2.8.

For all masonry *components* other than walls bending in the out-of-plane sense and masonry structures expected to remain essentially elastic, maximum reinforcement is limited in accordance with a prescribed strain distribution based on a tensile strain equal to five times the yield strain for the reinforcing bar closest to the edge of the member, and a maximum masonry compressive strain equal to 0.0025 for concrete masonry or 0.0035 for clay-unit masonry. By limiting longitudinal reinforcement in this manner, inelastic curvature capacity is easily depicted as the slope of the strain distribution.

Because axial force is implicitly considered in the determination of maximum longitudinal reinforcement, inelastic curvature capacity can be relied on no matter what the level of axial compressive force. Thus, the capacity reduction factors, ϕ, for axial load and flexure can be the same as for flexure alone. Also, confinement reinforcement is not required because the maximum masonry compressive strain will be less than ultimate values.

Calculated tensile force in the reinforcement is based on a stress equal to 1.25 times the yield stress to account for differences between the actual yield strength and the minimum specified strength, and the possibility of strain hardening. This increase of stress beyond yield also compensates for effects of discontinuous tensile strain fields that develop as a result of tensile cracking.

The numerical limits in the required provisions can be developed consistently from several approaches.

For structures expected to respond inelastically, one approach for the in-plane limits is based on the design model of a cantilever wall with a flexural hinge at the base. Neglecting elastic deformations of the wall and considering only the concentrated inelastic rotation at the hinge, the required inelastic curvature at the base of the wall is geometrically related to the maximum drift limits of Provisions Table 5.2.8. Assuming for simplicity that the length of the plastic hinge is equal to the plan length of the wall, the maximum drift ratio must equal the summation of the maximum usable compressive strain in the masonry plus the expected strain in the tensile reinforcement. Using a maximum usable compressive strain in the masonry between 0.0025 and 0.0035, the expected strain in the tensile reinforcement is about 4 times yield. The expected strain in the tensile reinforcement must somewhat exceed this, however, because the inelastic rotation is not concentrated at a single point, and because the inelastic hinge length can be less than the plan length of the wall. The required expected strain is therefore set at 5 times yield.

For structures expected to respond inelastically, the masonry compressive force is estimated using a rectangular stress block defined with parameters based on recent research done with the Technical Coordinating Committee for Masonry Research (TCCMaR).

For walls bending out of plane, the limit on maximum reinforcement is relaxed by considering a strain distribution based on 1.3 times the yield strain for the reinforcing bar closest to the member edge. This limiting strain distribution is less severe than that adopted for in plane bending. It is based on research done by Blondet and Mayes (1991).

For structures intended to undergo significant inelastic response, the Provisions are a technically sound way of achieving the design objective of inelastic deformation capacity. They are, however, unnecessarily restrictive for those structures not required to undergo significant inelastic deformation under the design earthquake.

Because unreinforced masonry structures, which have traditionally been regarded as non-ductile, are assigned an R value of 1.5, that value was taken as corresponding to essentially elastic response. Structures designed using an R value of 1.5 would be expected to reach but not exceed a critical strain condition corresponding to the development of yield strain in the extreme tensile reinforcement, and the development of the maximum useful compressive strain in the extreme compressive fiber of the masonry.

Because of the possibility of overstrength, yield strain was taken as 1.25 f_y. To allow a prudent margin of safety against limited inelastic behavior, the maximum strain in the tensile reinforcement was increased from 1.25 ε_y to 2 ε_y. The criterion for essentially elastic structures applies to both in- and out-of-plane flexure.

At the curvatures corresponding to yield strain or slightly past yield strain, the corresponding stress distribution in the masonry would be linear rather than an equivalent rectangle. Using that stress distribution, the axial load in the element, and the corresponding elastic stresses in tensile and compressive reinforcement, the maximum reinforcement can be calculated.

Maximum reinforcement per the requirements of Sec. 11.6.2.2.1 for an in-plane wall with uniformly distributed vertical reinforcement can be derived using simple equilibrium concepts to give:

$$\rho_{max} = \frac{0.64 f'_m \alpha - \dfrac{P_g}{bd}}{1.25 f_y (1 - \alpha) - 0.5 f_{s\,max} \alpha} \qquad \text{(C11.6.2.2-1)}$$

where ρ_{max} is the total amount of vertical steel divided by b and d; b is the width of the section; d is the distance from the extreme compressive fiber to the location of the tensile vertical bar closest to the edge of the member; α is equal to the depth of the compression zone divided by the effective depth, d; P_g is equal to the unfactored gravity compressive force; f_y is the specified yield stress of the reinforcement, and $f_{s\,max}$ is the maximum compressive stress in the vertical reinforcement.

Similarly, maximum reinforcement per the requirements of Sec. 11.6.2.2.2 for an out-of-plane wall with a single layer of vertical reinforcement centered on the wall section reduces to:

$$\rho_{max} = \frac{0.64 f'_m \alpha - \dfrac{P_g}{bd}}{2.50 f_y} \qquad \text{(C11.6.2.2-2)}$$

where $\rho_{g\,max}$ is the total amount of vertical steel divided by the gross area of the wall section.

For structures expected to undergo significant inelastic response, Equation C11.6.2.2-2 shows that for any maximum permitted reinforcement ratio, the Provisions directly limit the summation of forces from axial load and tensile reinforcement. Given prescriptive requirements for minimum reinforcement, the Provisions limit the axial load to a value that increases with the specified compressive strength of the masonry. This limitation may govern the maximum number of stories.

These maximum reinforcement requirements apply to all flexural members (walls, columns and beams) in all seismic design categories, for all structures expected to undergo significant inelastic response. Maximum reinforcement requirements for structures not expected to undergo significant elastic response, while similar in nature, impose less stringent limitations on maximum reinforcement.

For calibration purposes, maximum longitudinal reinforcement per Sec. 2108.2.3.3 of the 1997 Uniform Building Code is also plotted in Figures C11.6.2.2-1 and C11.6.2.2-2. The UBC criterion limits longitudinal reinforcement to no more than one-half of that resulting in a balanced condition where ultimate masonry compressive stress is equal to 0.003 and reinforcement is at its yield strain. A rectangular stress block is to be used with a stress equal to 0.85 times f'_m and a stress block depth equal to 0.85 times the compressed zone. No increase in the yield stress is specified by the UBC to account for increases due to higher expected strengths, strain hardening, or flexural cracking. The UBC criterion also considers axial force when limiting maximum reinforcement. However, in addition to gravity forces, axial forces due to earthquake effects times a load factor of 1.4 are also considered. Using the same procedure as used to derive the two former equations, the UBC criterion reduces to:

$$\rho_{g\,max} = \frac{1}{2}\left[\frac{0.723 f'_m \alpha - \dfrac{P_u}{bd}}{2.00 f_y}\right] \tag{C11.6.2.2-3}$$

where P_u is the factored axial load (1.0D + 1.0L +1.4E).

The UBC criterion results in a more restrictive limit on maximum reinforcement for axial compressive stress less than 381 psi for clay-unit masonry and 103 psi for concrete masonry (with the assumed values of f'_m). For axial compressive stresses above these values, the in-plane criterion per Sec. 11.6.2.2 results in a more restrictive limit on maximum reinforcement.

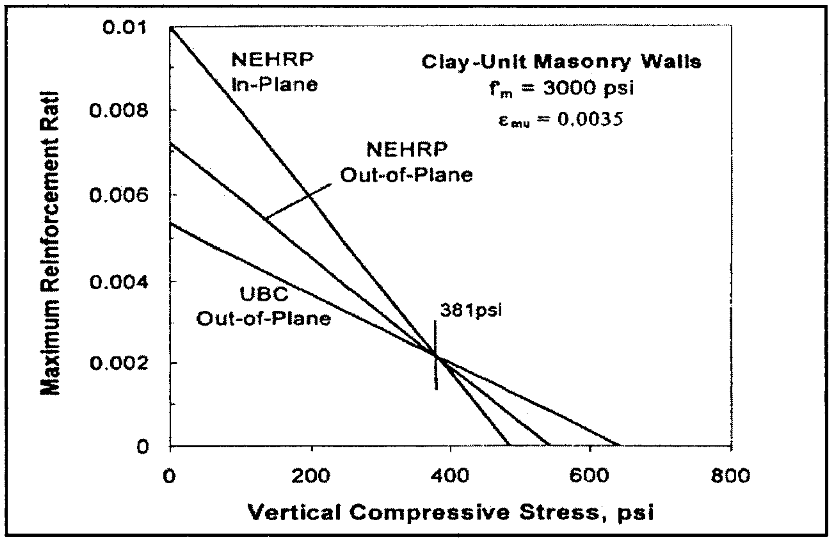


FIGURE C11.6.2.2-1 Maximum reinforcement for clay-unit masonry walls.

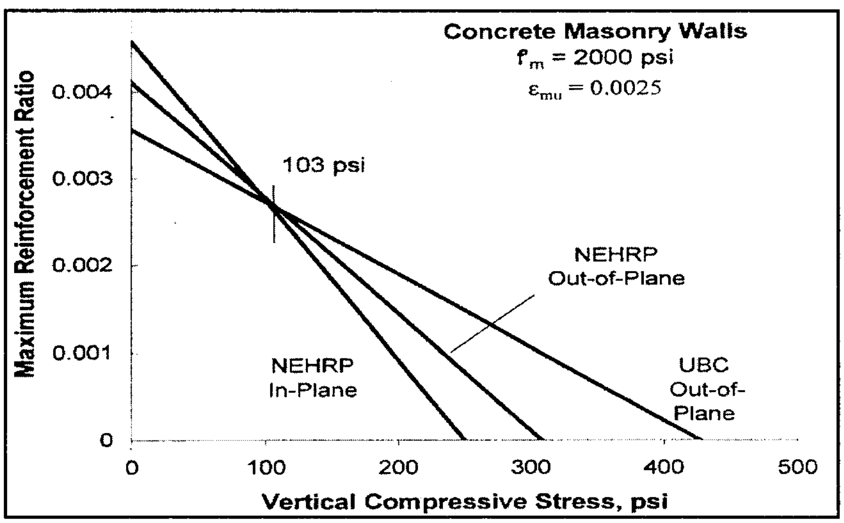


FIGURE C11.6.2.2-2 Maximum reinforcement for concrete masonry walls.

For further discussion, see He and Priestley (1992), Leiva and Klingner (1991), Limin and Priestley (1988), Merryman et al. (1989), Seible et al. (1992), and Shing et al. (1991).

11.6.3 Design of Plan (Unreinforced) Masonry Members:

11.6.3.5: The axial load strengths given by Eq. 11.6.3.5-1 and 11.6.3.5-2 are based on analysis of the results of axial load tests performed on clay and concrete masonry elements. For members having an h/r ratio not exceeding 99, the specimens failed at loads less than the Euler buckling load. Eq. 11.6.3.5-1 was empirically fit to test data for these members. For h/r values in excess of 99, the limited test data is adequately approximated by the Euler buckling equation.

11.7 SHEAR:

11.7.3 Design of Reinforced Masonry Members: The development of strength design procedures for masonry requires a reasonably simplified and accurate equation that is capable of predicting the ultimate shear strength of a masonry wall. Once agreed upon, this equation, together with appropriate ϕ factors, will form a key part of strength design procedures.

Over the past two decades many hundreds of tests have been performed in the U.S., Japan and New Zealand to determine the strength and ductility of concrete block and clay brick shear walls subjected to cyclic lateral load patterns. From these tests come equations to predict the shear strength of walls usually calibrated to the tests carried out by the particular researcher. Fattal and Todd (1991) compared the predictions of four different equations with available experimental results. The only flaw in this work was that they included the UBC design equations with the inference that the UBC equations were predictive equations for the ultimate shear strength of masonry. This is not the intent of the UBC equations. They were developed and then modified as part of the code development process to provide a lower bound on the shear capacity of masonry walls. Two other reports/papers were reviewed as part of preparing this overview document; Blondet et al. (1989) and Anderson and Priestley (1992) also looked at predictive equations which were more simplified than those included in the Fattal and Todd review. As a consequence, a total of six different predictive methods have been reviewed.

In summary, the methods include two or more of the following components:

$$V_u = V_m + V_{sh} + V_{sv} + V_p \qquad \text{(C11.7.3-1)}$$

where:

V_m = contribution of the masonry

V_{sh} = contribution of the horizontal steel

V_{sv} = contribution of the vertical steel

V_p = contribution of the axial load

The report by Fattal and Todd (1991) is quite thorough and the test data used to assess the Shing, Matsamura, and Architectural Institute of Japan (AIJ) predictive equations were also used to assess the methods proposed by Blondet et al. (1989) and Anderson and Priestley (1992) and the final TCCMaR equations that were developed as part of the TCCMaR study. The form of these equations are given in Table C11.7.3-2. Rather than present the details of each of the test results that were developed, a statistical summary is provided in Table C11.7.3-1. This provides the overall average, standard deviation and coefficient of variation for all 62 tests included in the Fattal and Todd report. The values given in Table C11.7.3-1 are the ratio of the shear strength obtained by the predictive equation divided by the ultimate strength obtained from the test. A perfect prediction has a ratio of 1 and a conservative prediction has a ratio less than 1.

TABLE C11.7.3-1

Tests	Shing	Okamoto	Matsamura	Blondet et al.	Anderson & Priestley	TCCMaR
All 62 tests						
Mean	0.83	0.81	0.91	1.03	1.06	1.02
Standard Deviation	0.23	0.27	0.20	0.24	0.23	0.24
Coefficient of Variation	0.05	0.07	0.04	0.06	0.05	0.05
Mean Values						
Tests 1-10 (Shing)	0.94	1.25	0.93	0.88	1.02	0.87
Tests 11-27 (Matsu-mura)	0.89	0.82	0.99	1.10	1.13	1.07
Tests 27-38 (Okamoto)	0.65	0.76	0.75	0.80	0.86	0.81
Tests 39-62 (Sveinsson)	0.82	0.66	0.91	1.13	1.11	1.12

Also included in Table C11.7.3-1 are the mean values of the four different sets of tests. Test 1-10 are from Shing et al. (1991), Tests 11-28 are from Matsamura (1987), Tests 29-37 are from Okamoto et al. (1987), and Tests 38-62 are from Sveinsson et al. (1985).

TABLE C11-7.3-2

	Masonry Component	Horizontal Steel	Vertical Load
TCCMAR	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Blondet et al.	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$\sqrt{V_m^2+\frac{V_m\sigma_c}{1.5}}-V_m$
Anderson	$2.9\sqrt{f_m'}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Shing[a]	$(0.0217\rho_v f_v + 0.166)\sqrt{f_m'}$	$\left(L-\frac{2d^1}{S_h}-1\right)\frac{S_h}{L}\rho_h f_h$	$(0.0217\sigma_c)\sqrt{f'_m}$

TCCMAR	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Matsamura[a]	$\left[\left(\frac{0.76}{r_d+0.7}+0.012\right)\left(4.04\rho_v^{0.3}\right)\sqrt{f'_m}\right]\frac{d}{L}$	$\left[0.01575(\rho_h f_h)^{1/2}\sqrt{f'_m}\frac{\delta d}{L}\right]$	$(0.175\sigma_c)\frac{d}{L}$
AIJ	$\left[4.64\rho_v^{0.23}(0.01f'_m+0.176)\left(\frac{1}{R_c+0.12}\right)\right]\frac{d}{L}$	$\left[0.739(\rho_N f_h)^{1/2}+0.739(\rho_v f_v)^{1/2}\right]\frac{d}{L}$	$(0.0875\sigma_c)\frac{d}{L}$

[a]These equations are in metric units.

As part of the TCCMaR studies, it was decided to use a combination of the Blondet et al. and Anderson and Priestley equations. In comparing the manner in which the two methods account for contribution of the masonry component, it was decided to use the Blondet form. As part of the Berkeley tests (Mayes et al., 1976, Chen et al., 1978, Hidalgo et al., (1978, 1979), it was concluded that the M/Vd ratio should be part of the masonry equation rather than just a straight function of 2.9 $\sqrt{f'_m}$ as in the Anderson and Priestley equation. Furthermore, there was very little numerical difference in the values used to account for the vertical load contribution. As a consequence, it was decided to use the more simplified form of $0.25\sigma_c$ used by Anderson and Priestley. The final form of the TCCMaR equation was given as:

$$v = (4 - 1.75M/Vd)\sqrt{f'_m} + 0.5\rho_h f_{yh} + 0.25\sigma_c \quad \text{(C11.7.3-2)}$$

The metric equivalent of Eq. C11.7.3-2 is:

$$v = 0.083(4 - 1.75M/Vd)\sqrt{f'_m} + 0.5\rho_h f_{yh} + 0.25\sigma_c$$

Some members of TCCMaR believed that some contribution of vertical steel should be included and this issue was investigated. Many of the test specimens only included jamb steel and consequently two different vertical steel contributions were investigated: $1/4\rho_v f_{yv}$ and $1/4\rho_{vi} f_{yvi}$ where ρ_v is the total vertical steel and ρ_{vi} is only the interior vertical steel and neglects the jamb steel. The correlation and the test results were not as good when a contribution from vertical steel was included and consequently it was decided not to include it in the recommended TCCMaR shear equation.

Application of the shear strength equation to partially grouted masonry was based in part on Fattal (1993a and 1993b).

11.8 SPECIAL REQUIREMENTS FOR BEAMS:

11.8.1: Masonry beams may be loaded normal to their plane by wind or earthquake forces. The beam must have adequate strength to span between support points under the action of the out-of-plane loads. The arbitrary limits of 50 and 32 were judged to be adequate absolute limits on the unbraced span to beam width ratios for the conditions listed.

11.8.2: Gravity loading of a masonry beam may be applied eccentrically to its vertical centroidal plane. The lateral supports of the masonry building should restrain the beam from rotation under the eccentric action of the gravity load.

If the beam is supported laterally at one edge only (top or bottom), then the lateral support should have the moment capacity to restrain the rotation caused by loading normal to the face of the beam that is eccentric to the support point.

11.8.3: A minimum amount of flexural reinforcement in the positive moment zone of the beam is specified. This minimum is specified as a ratio, ρ, of the quantity of the reinforcement to the cross-sectional area of the beam. The minimum ratio specified is intended to require that the post-cracked moment capacity exceeds the uncracked moment capacity of the section.

These requirements for a minimum quantity of positive moment reinforcement assumes that cracking has occurred in zones of negative moment and that the change in beam stiffness has increased the positive moment. However, if the positive moment capacity of the reinforced section exceeds the uncracked positive moment capacity, transfer of moment to this zone is accommodated.

If a section of the adjacent concrete floor serves as the compression flange of the beam, minimum reinforcement is based on the masonry section which is in tension due to positive moment.

11.8.4 Deep Flexural Members: The theory used for design of beams has a limited applicability to deep beams. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stress in the body of the deep beam requires that deep beam theory be used for design of members that exceed the specified limits of span to depth ratio. Analysis of wall sections that are used as beams generally will result in a distribution of tensile stress that requires the lower one-half of the beam section to have uniformly distributed reinforcement. The uniform distribution of reinforcement resists tensile stress caused by shear as well as flexural moment.

The flexural reinforcement for deep beams must meet or exceed the minimum flexural reinforcement ratio of Sec. 11.8.3. Additionally, horizontal and vertical reinforcement must be distributed throughout the length and depth of deep beams and must provide reinforcement ratios of at least 0.001. Distributed flexural reinforcement may be included in the calculations of the minimum distributed reinforcement ratios.

Flexural reinforcement that is lumped entirely at the bottom and/or top of a deep flexural member, however, should be ignored when calculating the distributed horizontal reinforcement ratio. In such a case, the lumped flexural steel must provide a minimum flexural reinforcement ratio of $120/f_y$ in accordance with Sec. 11.8.3. For Grade 60 steel, this requirement is equivalent to a minimum flexural reinforcement ratio of 0.002.

Although this flexural reinforcement ratio results in twice the ratio required by Sec. 11.8.4.3, the flexural steel is lumped at the top and/or bottom of the beam and is not uniformly distributed. Since the intent of Sec. 11.8.4.3 is to ensure a minimum quantity of uniformly distributed reinforcement throughout the depth of the deep beam, the lumped flexural steel is not considered when calculating the minimum distributed reinforcement ratios.

11.9 SPECIAL REQUIREMENTS FOR COLUMNS:

11.9.1: Maximum and minimum limitations on the area of longitudinal reinforcement for columns are traditional values that have been in codes for many years. Minimum areas are limited so that creep of the masonry, which tends to transfer load from masonry to reinforcing steel will not result in increasing the stress in the steel to yield level. The maximum area limitation represents a practical limit on the amount of reinforcing steel in terms of economy and steel placement. No testing or research has been done to justify changes in these traditional values.

11.9.2: The minimum number of bars in columns also is a traditional number. It is obviously appropriate, however, to suit rectangular or square column shapes and tying requirements.

11.9.3: The lateral tie restrictions in this section are also traditional. The column tie bending requirements of Part c are to be as shown.

Reinforcement is restricted to an amount below the area required for flexural bending only in order to preserve a ductile failure condition (i.e., steel will reach ultimate yield strain before concrete reaches ultimate yield strain which would be defined as a brittle failure). It is therefore important to keep the reinforcement ratio low.

11.10 SPECIAL REQUIREMENTS FOR SHEAR WALLS:

11.10.1 through 11.10.5: Detailing requirements for masonry shear walls have been reorganized for 1997 in Sec. 11.10.1 through 11.10.5 to provide direct correlations with those categories given as line items in Table 5.2.2: ordinary plain masonry shear walls, detailed plain masonry shear walls, ordinary reinforced masonry shear walls, intermediate reinforced masonry shear walls, and special reinforced masonry shear walls. This was done so that variable R, Ω, and C_d factors could be given for each shear wall category rather than specifying detailing requirements per the Seismic Design Category as was done in previous editions of the Provisions. This reorganization is more consistent with the other material chapters, which are organized by type of lateral-force-resisting elements (e.g., ordinary, intermediate, or special moment resisting frames).

The word "plain" refers to the condition when a wall is unreinforced or tensile stresses in reinforcement, if any, are neglected. The word "reinforced" refers to the condition when tensile stresses in reinforcement are considered in the design process. "Detailed plain" and "intermediate reinforced" walls much have minimum reinforcement per Seismic Design Category C whereas "ordinary plain" and "ordinary reinforced" walls do not need to have any minimum reinforcement. Reinforcement requirements for "special reinforced" walls follow the requ9irements for Seismic Design Categories D and E. Requirements in each Seismic Design Category that are not germane to masonry walls have bene retained in Sec. 11.3.5 through 11.3.9. in newly.

11.10.6 Flanged Shear Walls: Tests on flanged shear walls (Priestley and Limin, 1990; Sieble et al., 1992) have indicated that if the conditions of Sec. 11.10.3.1 are satisfied, the flange will act in conjunction with the web as a part of the flexural member.

The tributary flange widths defined in Sec. 11.10.3.3 and 11.10.3.4 are considered to be values appropriate for predicting flexural behavior and strength. The values were taken from experimental results. This has significance when calculating probable shear force on the wall, which is related to the probable maximum flexural strength. For the calculation of maximum allowable reinforcement ratios, the reinforcement in the flange of the width specified in Sec. 11.10.3.4 must be considered as part of the maximum reinforcement ratio.

11.10.7 Coupled Shear Walls: Coupled shear walls are defined as shear walls in a common wall plane that are interconnected or coupled by spandrel beams. These beams are typically at each floor level. The coupling beams can be a section of a reinforced concrete floor that has continuity with the shear walls. Caution should be exercised to distinguish between coupled shear walls and walls with openings. In a coupled wall system, the yield limit state is allowed only in the coupling beam and at the base of the shear wall. If the flexure or shear yield state occurs in the wall between coupling beams, the system is a wall with openings. This system has very limited ductility and should be redesigned to prevent yielding in the reinforced wall at points other than the base of the shear wall.

Conformance with the requirement that the coupling beams reach their moment limit state at or before the shear wall reaches its moment limit state need not be checked if the ratio of the depth of the shear wall to the depth of the coupling beams exceeds 3 or more and the length of the coupling beams is less than one-half of the story height. Linear elastic analyses of the coupled wall system are inadequate to determine the yield status of the shear wall and the coupling beams. The stiffness of the shear wall will degrade rapidly in the first story. The shear walls in the upper stories may be uncracked.

11.10.7.2 Shear Strength of Coupling Beams: The nominal shear strength of coupling beams must be equal to the shear caused by development of a full yield hinge at each end of the coupling beams. This nominal shear strength is estimated by dividing the sum of the calculated yield moment capacity of each end of the coupling beams, M_1 and M_2, by the clear span length, L.

A coupling beam may consist of a masonry beam and a part of the reinforced concrete floor system. Reinforcement in the floor system parallel to the coupling beam should be considered as a part of the coupling beam reinforcement. The limit of the minimum width of floor that should be used is six times the floor slab thickness. This quantity of reinforcement may exceed the limits of Sec. 11.6.2.2 but should be used for the computation of the normal shear strength.

11.12 GLASS-UNIT MASONRY AND MASONRY VENEER: Chapters 11 and 12 of ACI 530-95/ASCE 5-95/TMS 402-95 have been newly introduced in the 1997 Provisions to address design of glass-unit masonry and masonry veneer. Direct reference is made to these chapters for design requirements. Investigations of seismic performance have shown that architectural *components* meeting these requirements perform well (Jalil, Kelm and Klingner, 1992, and Klingner, 1994).

REFERENCES

Abboud, B. E., X. Lu, and F. C. Schmitt. 1993. "An Evaluation of Three Current Deflection Methods for Predicting the Lateral Deflection of Masonry walls." In Proceedings of the Sixth North American Masonry Conference, Philadelphia, Pennsylvania, pp.73-85.

Abboud, B. E., and A. A. Hamid. 1987. "A Comparative Study of the Flexural Behavior of Reinforced Block Masonry and Reinforced concrete Using Small Scale Model walls." In Proceedings of the Fourth North American Masonry Conference, Los Angeles, California.

Abboud, B. E., A. A. Hamid, and H. G. Harris. 1990. "Small Scale Modeling of Concrete Block Masonry structures," ACI Structural Journal 87(2):145-155.

Abboud, B. E. 1987. The Use of Small Scale Direct Models for Concrete Block Masonry Assemblages and Slander Reinforced walls Under Out-of-Plane Loads, Doctoral Thesis, Drexel University, Philadelphia, Pennsylvania.

Blondet, J. M., R. L. Mayes, T. E. Kelley, R. R. Villablanca, and R. E. Klingner. 1989. Performance of Engineered Masonry in the Chilean Earthquake of March 3, 1985: Implications for U.S. Design Practice. Austin: Phil Ferguson Structural Engineering Laboratory, University of Texas.

Brown, Russell H., and A. Rhett Whitlock. 1983. "Strength of Anchor Bolts in Grouted Masonry," ASCE Journal of Structural Engineering 109(6) June.

CEB. 1995. "Design of Fastenings in Concrete (Draft CEB Guide, Parts 1 to 3)" and "Fastenings for Seismic Retrofitting (State-of-the-Art Report on Design and Application)," Task Group 3.5 (Embedments), Euro-International Concrete Committee (CEB), CEB Bulletin D'Information No. 226, Comite Euro-International du Beton, August.

Chen, S. W., P. A. Hidalgo, R. L. Mayes, R. W. Clough, and H. D. McNiven. 1978. Cyclic Loading Tests of Masonry Single Piers, Vol. 2, EERC Report UCB/EERC-78/27. Berkeley: University of California Earthquake Engineering Research Center.

Fattal, S. G. 1993a. strength of Partially Grouted Masonry shear walls, Report NIST 5147. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G. 1993b. The Effect of Critical Parameters on the Behavior of Partially Grouted Masonry shear walls Under Lateral Loads, NIST Report NISTIR 5116. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G., and D. R. Todd. 1991. Ultimate strength of Masonry walls: Prediction vs. Test Results, NIST Report NISTIR 4633. Gaithersburg, Maryland: National Institute of Standards and Technology.

Hamid, A. A., M. K. Hatem, H. G. Harris, and B. E. Abboud. 1990. Hysteretic Response and Ductility of Reinforced concrete Masonry walls Under Out-of-Plane Loading." In Proceedings of the Fifth North American Conference, Urbana, Illinois, pp. 397-410.

Hamid, A. A., B. E. Abboud, M. W. Farah, M. K. Hatem, and H. G. Harris. 1989. "Response of Reinforced Block Masonry walls to Out-of-Plane Static Loads," in U.S.-Japan Coordinated Program For Masonry Building Research Report 3.2(a), Drexel University.

He, L., and N. Priestley. 1992. Seismic Behavior of Flanged Masonry shear walls - Final Report, TCCMaR Report 4.1-2.

Hidalgo, P. A., R. L. Mayes, H. D. McNiven, and R. W. Clough. 1978. Cyclic Loading Tests of Masonry Single Piers, Vol. 1 and 3, EERC Report UCB/EERC-78/27 and 79/12. Berkeley, University of California Earthquake Engineering Research Center.

Horton, R. T., and M. K. Tadros. 1990. "Deflection of Reinforced Masonry Members," ACI Structural Journal 87(4):73-85.

Jail, I., W. Kelm, and R. E. Klingner. 1992. Performance of Masonry and Masonry Veneer Buildings in the Loma Prieta Earthquake. PMSEL Report 92-1. Austin: University of Texas, Department of Civil Engineering.

Klingner, R. E. 1994. Performance of Masonry Structures in the Northridge, California, Earthquake of January 17, 1991. Boulder, Colorado: The Masonry Society.

Leiva, G., and R. Klingner. 1991. In-Plane Seismic Resistance of Two-story Concrete Masonry shear walls with Openings, TCCMaR Report 3.1(c)-2.

Limin, H., and N. Priestley. 1988. Seismic Behavior of Flanged Masonry shear walls, TCCMaR Report 4.1-1.

Malik, J. B., J. A. Mendonca, and R. E. Klingner. 1982. "Effect of Reinforcing Details on the Shear Resistance of Short Anchor Bolts Under Reversed Cyclic Loading," Journal of the American Concrete Institute, Proceedings, Vol. 79, No. 1, January-February, pp 3-11.

Matsumura, A. 1987. "Shear strength of Reinforced Hollow Unit Masonry walls." In Proceedings of the 4th North American Masonry Conference, Los Angeles, California.

Mayes, R. L., Y. Omote, and R. W. Clough. 1976. Cyclic Shear Tests of Masonry Piers, Vol. 1, Report 76-8. Berkeley: University of California Earthquake Engineering Research Center.

Merryman, K., G. Leiva, N. Antrobus, and R. Klingner. 1989. In-Plane Seismic Resistance of Two-story Concrete Masonry Coupled Shear Walls, TCCMaR Report 3.1(c)-1.

Okamoto, S., Y. Yamazaki, T. Kaminosono, M. Teshigawara, and H. Hirashi. 1987. "Seismic Capacity of Reinforced Masonry walls and Beams in Wind and Seismic Effects." In Proceedings of the 18th joint Meeting, U.S. - Japan Panel on Wind and Seismic Effects, ed. by N. J. Raufaste, Report NBSIR 87-3540. Gaithersburg, Maryland: National Institute of Standards and Technology.

Priestley, N., and H. Limin. 1990. "Seismic Response of T-Section Masonry shear walls." TMS Journal 9(1):10-19.

Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi. 1992. The U.S. - TCCMaR Five-story Full Scale Masonry Research Building Test - Preliminary Report, TCCMaR Report 9.2-4.

Shaikh, A. Fattah. 1996. "Design of Hooked Anchor bolts in Concrete and Masonry: Proposed Code Provisions and Commentary," prepared for the National Codes and Standards Council, 1996.

Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller. 1991. Response of Single-story reinforced masonry shear walls to In-Plane Lateral Loads, TCCMaR Report 3.1(a)-2.

Shing, P. B., M. Schuller, and V. S. Hoskere. 1990. "In-Plane Resistance of Masonry shear walls," ASCE Journal of Structural Engineering 116(3).

Sveinsson, B. I., H. D. McNiven, and H. Sucuoglu. 1985. Cyclic Shear Tests of Masonry Piers, Vol 4, Report UCB/EERC 85-15. Berkeley: University of California Earthquake Engineering Research Center.

Wolde-Tinsae, A. M., R. H. Atkinson, and A. A. Hamid. 1993. "State-of-the-Art Modulus of Elasticity of Masonry." In Proceedings of the Sixth North American Masonry Conference. Boulder, Colorado: The Masonry Society.

The following Technical Coordinating Committee for Masonry Research task reports not specifically cited providing the substantiating data for the strength design criteria presented in this chapter are available through the Earthquake Engineering Research Center Library in Richmond, California (phone 415-231-9403):

Task No. Author(s) and Title

1.1-1: Atkinson and Kingsley, Comparison of the Behavior of Clay & Concrete Masonry in Compression, September 1985. 151 pgs.

1.2(a)-1: Hamid, A. A., G. F. Assis, and H. G. Harris, Material Models for Grouted Block Masonry, August 1988. 67 pgs.

1.2(a)-2: Assis, G. F., A. A. Hamid, and H. G. Harris, Material Models for Grouted Block Masonry, August 1989. 134 pgs.

1.2(b)-1: Young, J. M., and R. H. Brown, Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient, May 1988, 170 pgs.

1.3-1: Atkinson, R. H., An Assessment of Current Material Test Standards for Masonry Limit States Design Methods, June 1991. 38 pgs.

2.1-1: Hart, G., and M. Basharkhah, Slender Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01), September 1987. 68 pgs.

2.1-2: Hart, G., and M. Basharkhah, Shear Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01). September 1987. 75 pgs.

2.1-3: Nakaki, D., and G. Hart, Uplifting Response of Structures Subjected to Earthquake Motions, August 1987. 200 pgs.

2.1-4: Hart, G., N. Sajjad, and M. Basharkhah, Inelastic Column Analysis Computer Program (INCAP, Version 1.01), March 1988.

2.1-5: Hong, W. K., G. C. Hart, and R. E. Englekirk, Force-Deflection Evaluation and Models for University of Colorado Flexural walls, December 1989.

2.1-6: Hart, G. C., J. W. Jaw, and Y. K. Low, SCM Model for University of Colorado Flexural Walls, December 1989. 31 pgs.

2.1-7: Hart, G. C., N. Sajjad, and M. Basharkhah, Inelastic Masonry Flexural Shear Wall Analysis Computer Program, February 1990. 41 pgs.

2.1-8: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, DPC Gymnasium, Elastic Time History Analysis Using SAP90, February 1992. 41 pgs.

2.1-9: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, TMS Shopping Center, Elastic Time History Analysis Using SAP90, February 1992. 42 pgs.

2.1-10: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and D. J. Drag, Seismic Performance Study, RCJ Hotel, February 1992. 51 pgs.

2.1-11: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, Performance Study, 2-Story Masonry Wall-Frame Building, February 1992. 112 pgs.

2.1-12: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and J. D. Drag, Seismic Performance Study, Designed by Tentative Limit Sates Design Standard, February 1992. 75 pgs.

2.2-1: Ewing, R. D., A. El-Mustapha, and J. Kariotis, FEM/I - A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components, December 1987 (Revised June 1990). 124 pgs.

2.2-2: Ewing, R. D., Parametric Studies on Reinforced Masonry Shear Walls Using FEM/I, A Nonlinear Finite Element Analysis Program, March 1992.

2.2-3: Ewing, R. D., Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard, March 1992. 48 pgs.

2.3-1: Ewing, R., J. Kariotis, and A. El-Mustapha, LPM/I, A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, August, 1987. 200 pgs.

2.3-2: Kariotis, J., A. El-Mustapha, and R. Ewing, Influence of Foundation Model on the Uplifting of Structures, July 1988. 50 pgs.

2.3-3: Kariotis, J., A. Rahman, and A. El-Mustapha, Investigation of Current Seismic Design Provisions for Reinforced Masonry Shear Walls, January 1990. 48 pgs.

2.3-4: Kariotis, J., A. Rahman, O. Waqfi, and R. Ewing, Version 1.03 LPM/I - A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, February 1992. 227 pgs.

2.3-5: Kariotis, J., O. Waqfi, and R. Ewing, R., A Computer Program Using Beam Elements for the Nonlinear, Dynamic Analysis of Lumped Parameter Models, February 1992. 96 pgs.

2.3-6: Kariotis, J., and O. Waqfi, Comparison of the Dynamic Response of a Damped MDOF Nonlinear Beam Model with an Equivalent SDOF Hysteretic Model, April 1992. 88 pgs.

2.3-7: Kariotis, J., and O. Waqfi, Recommended Procedure for Calculation of the Balanced Reinforcement Ratio, February 1992. 73 pgs.

2.4(b)-1: Button, M. R., and R. L. Mayes, Out-of-Plane Seismic Response of Reinforced Masonry Walls: Correlation of Full-Scale Test and Analytical Model Results, March 1991. 65 pgs.

3.1(a)-1: Scrivener, J., Summary of Findings of Cyclic Tests on Masonry Piers, June 1986. 7 pgs.

3.1(a)-2: Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller, Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads, January 1991. 136 pgs.

3.1(b)-1: Seible, F., and H. LaRovere, Summary of Pseudo Dynamic Testing, February 1987. 46 pgs.

3.1(b)-2: Igarashi, A., F. Seible, and G. Hegemier, Development of the Generated Sequential displacement Procedure and the Simulated Seismic Testing of the TCCMaR Three-Story In-Plane Walls, June 1993.

3.1(c)-1: Merryman, K., G. Leiva, B. Antrobus, and R. Klingner, In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled shear walls, September 1989. 176 pgs.

3.1(c)-2: Leiva, G., and R. Klingner, In-plane Seismic Resistance of Two-story Concrete Masonry Shear Walls with Openings, August 1991. 326 pgs.

3.2(a)-1: Hamid, A., B. Abboud, M. Farah, K. Hatem, and H. Harris, Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads; September 1989. 120 pgs.

3.2(b)-1: Agbabian, M., S. Adham, S. Masri, V. Avanessian, and V. Traina, Out-of-Plane Dynamic Testing of Concrete Masonry Walls, Vol. 1 and 2, July 1989. 220 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 1: General Information, April 1991. 172 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 2: Walls No. 4 and 6 (Group 1), April 1991. 267 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 3: Walls No. 8, 9, 10 and 11 (Group 2), April 1991. 310 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes, Vol. 4: Walls No. 3, 5, and 7 (Group 3), April 1991. 256 pgs.

4.1-1: He, L., and M. J. N. Priestley, Seismic Behavior of Flanged Masonry Shear Walls, May 1988. 119 pgs.

4.1-2: He, L., and M. J. N. Priestley, Seismic Behavior of Flanged Masonry Shear Walls - Final Report, November 1992. 279 pgs.

4.2-1: Hegemier, G., and H. Murakami, On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part I: Experimental.

4.2-2: Hegemier, G., and H. Murakami, On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part II: Theoretical.

5.1-1: Porter, M., and A. Sabri, Plank Diaphragm Characteristics, July 1990. 226 pgs.

5.2-1: Porter, M., F. Yeomans, and A. Johns, Assembly of Existing Diaphragm Data, July 1990. 142 pgs.

6.2-1: Scrivener, J., Bond of Reinforcement in Grouted Hollow-Unit Masonry: A State-of-the-Art, June 1986. 53 pgs.

6.2-2: Soric, Z., and L. Tulin, Bond Splices in Reinforced Masonry, August 1987. 296 pgs.

7.1-1: Paulson, T., and D. Abrams, Measured Inelastic Response of Reinforced Masonry Building structures to Earthquake Motions, October 1990. 294 pgs.

8.1-1: Hart, G., A Limit State Design Method for Reinforced Masonry, June 1988.

8.1-2: Hart, G., Expected Value Design in the Context of a Limit Sate Design Methodology, February 1990.

8.2-1: Hart, G., and G. T. Zorapapel, Reliability of Concrete Masonry Wall Structures, December 1991. 229 pgs.

8.2-2: Hart, G., and N. Sajjad, Confinement in Concrete Masonry, December 1990.

8.2-3: Hart, G., and J. Jang, Seismic Performance of Masonry Wall Frames, December 1991.

9.1-1: Kariotis, J. C., and A. W. Johnson, Design of Reinforced Masonry Research Building, September 1987. 42 pgs.

9.1-2: Kariotis, J. C., and O. M. Waqfi, Trial Designs Made in Accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings, February 1992. 184 pgs.

9.2-1: Seible, F., Report on Large Structures Testing Facilities in Japan, September 1985. 120 pgs.

9.2-2: Seible, F., Design and Construction of the Charles Lee Powell Structural Systems Laboratory, November 1986. 65 pgs.

9.2-3: Seible, F., The Japanese Five-story Full Scale Reinforced Masonry Building Test, January 1988. 100 pgs.

9.2-4: Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi, The U.S. - TCCMaR Five-story Full Scale Masonry Research Building Test - Preliminary Report, October 1992. 58 pgs.

11.1-1: TCCMaR, Summary Report: U.S. Coordinated Program for Masonry Building Research, September 1985 to August 1986. 190 pgs.

11.1-2: TCCMaR, Status Report: U.S. Coordinated Program for Masonry Building Research, November 1988. 170 pgs.

Chapter 12 Commentary

WOOD STRUCTURE DESIGN REQUIREMENTS

12.1.2 REFERENCE DOCUMENTS: Wood construction practices have not been codified in a form that is standard throughout the country. A major change for the 1997 *Provisions* was the incorporation by reference of the *Load and Resistance Factor Standard for Engineered Wood Construction* (LRFD), ASCE 16. Engineered wood strength design as prescribed in the *Provisions* generally follows the LRFD specification (ASCE 16). Conventional light frame construction practice as prescribed in the *Provisions* generally follows the requirements of the *One- and Two-Family Dwelling Code*, CABO Code, jointly sponsored by the three model code organizations. The *One- and Two-Family Dwelling Code* is a revised and updated version of the Federal Housing Administration's (FHA) *Minimum Property Standards.*

APA N375B and PS 2 indicate that the term "structural-use panel" has replaced the term "plywood" and this change in terminology was reflected in the 1991 and 1994 *Provisions* and was continued in this 1997 edition. The term "structural-use panel" includes wood-based products manufactured to meet a performance standard (PS 2). One requirement of this performance standard is bracing or lateral force resistance capability. These products include oriented strand board (OSB), plywood, and composite panels. In the 2000 *Provisions*, "wood structural panel" replaces "structural-use panel."

Many wood frame structures are a combination of engineered wood and "conventional" light frame construction. Wood also is used in combination with other materials (American Institute of Timber Construction, 1985; Breyer, 1993; Faherty and Williamson, 1989; Hoyle and Woeste, 1989; Somayaji, 1992; Stalnaker and Harris, 1989). The requirements of the model building codes were used as a resource in developing the requirements introduced in the 1991 *Provisions* and further modified in this edition.

The general requirements of Chapter 12 cover construction practices necessary to provide a performance level of seismic resistance consistent with the purposes stated in Chapter 1. These requirements also may be related to gravity load capacity and wind force resistance which is a natural outgrowth of any design procedure.

For the 2000 *Provisions*, the reference documents continues to be grouped according to their primary focus into three subsections: Sec. 12.1.2.1, Engineered Wood Construction; Sec. 12.1.2.2, Conventional Construction; and Sec. 12.1.2.3, Materials Standards.

12.1.3 Notations: These variable definitions are included to assist the reader in understanding the equations and tables used in the chapter. To the extent possible, these definitions are compatible with the usage of the symbols in other chapters of the *Provisions* and ASCE 16. The definition of "factored resistance" has been added as the values of $\lambda\phi D$ to account for the time effect factor and resistance factor. This is the basis of all values presented in this chapter.

12.2 DESIGN METHODS: Prior to the publication of ASCE 16, typical design of wood frame *structures* followed the American Forest and Paper Association (AF&PA) *National Design Specification for Wood Construction* (NDS) (AF&PA, 1991). The NDS is based on "allowable" stresses and implied factors of safety. However, the design procedure provided by the *Provisions* was developed on the premise of the resistance capacity of members and connections at the yield level (ASCE, 1988; Canadian Wood Council, 1990 and 1991; Keenan, 1986). In order to accommodate this difference in philosophy, the 1994 and prior editions of the *Provisions* made adjustments to the tabulated "allowable" stresses in the reference documents.

With the completion of the *Load and Resistance Factor Standard for Engineered Wood Construction* (ASCE, 1995), the modifications and use of an "allowable" stress based standard is no longer necessary. Therefore, the 1997 *Provisions* includes the LRFD standard by reference (ASCE 16) and uses it as the primary design procedure for engineered wood construction. The use of ASCE 16 continues in the 2000 *Provisions*. In the 1997 provisions, the resistance shown in Tables 12.4.3-2a and b were reduced 10 percent to account for capacity reductions observed in cyclic testing of shear walls. (Dolan, 1996; Rose, 1996). This reduction was reviewed during the 2000 revision of the *Provisions* when additional test data were available and the decision was reversed and the resistance values returned to previous levels. However, the capacities provided for diaphragms were not reduced because the severe, repeated racking damage that occurred in shear walls has not been noted in diaphragms in recent earthquakes.

Conventional light-frame construction, a prescriptive method of constructing wood structures, is allowed for some performance categories. These structures must be constructed according to the requirements set forth in Sec. 12.5 and CABO Code. If the construction deviates from these prescriptive requirements, then the engineered design requirements of Sec. 12.3 and 12.4 and ASCE 16 shall be followed. If a structure that is classified as conventional construction contains some structural elements that do not meet the requirements of conventional construction, the elements in question can be engineered in accordance with Sec. 12.2.2.1 without changing the rest of the structure to engineered construction. The extent of design to be provided must be determined by the responsible registered design professional; however, the minimum acceptable extent is often taken to be force transfer into the element, design of the element, and force transfer out of the element. This does not apply to a structure that is principally an engineered structure with minor elements that could be considered conventional. When more than one braced wall line or diaphragm in any area of a conventional residence requires design, the nature of the construction may have changed, and engineered design might be appropriate for the entire lateral-force-resisting system. The absence of a ceiling diaphragm may also create a configuration that is non-conventional. The requirement for engineering portions of a conventional construction structure to maintain lateral-force resistance and stiffness is added to provide displacement compatibility. This is similar to the requirement in Sec. 12.3.3.

Alternate Strength of Members and Connections: It remains the intent of the *Provisions* that load and resistance factor design be used. When allowable stress design is to be used, however, the factored resistance of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, (ϕ), times 2.16 times the allowable stresses permitted in the *National Design Specification for Wood Construction* (NDS) and supplements (AF&PA, 1991). The allowable stresses used shall not include a duration of load factor, C_D. The value of the capacity reduction factor, ϕ, shall be as follows:

Wood members	
In flexure	$\phi = 1.00$
In compression	$\phi = 0.90$
In tension	$\phi = 1.00$
In shear and torsion	$\phi = 1.00$
Connectors	
Anchor bolts, bolts, lag bolts, nails, screws, etc.	$\phi = 0.85$
Bolts in single shear in members that are part of a seismic-force-resisting system	$\phi = 0.40$

These "soft" conversions from allowable stress design values to load and resistance factor design values appeared in Sec. 9.2 in the 1994 *Provisions*. For the 2000 *Provisions*, the factored resistance of shear walls and diaphragms shall be in accordance with Tables 12.4.3-1a and b and Tables 12.4.3-2a and b.

An alternative method of calculating soft conversions is provided in ASTM D5457-93. The reader is cautioned, however, that the loads and load combinations to be used for conversion are not specified so it is incumbent upon the user to determine appropriate conversion values.

12.3 GENERAL DESIGN REQUIREMENTS FOR ENGINEERED WOOD CONSTRUCTION: Engineered construction for wood *structures* as defined by the *Provisions* encompasses all structures that cannot be classified as conventional construction. Therefore, any structure exceeding the height limitations or having braced walls spaced at greater intervals than prescribed in Table 12.5.1-1 or not conforming to the requirements in Sec. 12.5 must be engineered using standard design methods and principles of mechanics. Framing members in engineered wood construction are sized based on calculated capacities to resist the loads and forces imposed. Construction techniques that utilize wood for lateral force resistance in the form of diaphragms or shear walls are discussed further in Sec. 12.4. Limitations have been set on the use of wood diaphragms that are used in combination with concrete and masonry walls or where torsion is induced by the arrangement of the vertical resisting elements. A load path must be provided to transmit the lateral forces from the diaphragm through the vertical resisting elements to the foundation. It is important for the registered design professional to follow the forces down, as for gravity loads, designing each connection and member along the load path.

Although wood moment resisting frames are not specifically covered in the *Provisions*, they are not excluded by them. There are several technical references for their design, and they have been used in Canada, Europe, and New Zealand. Wood moment resisting frames are designed to resist both vertical loads and lateral forces. Detailing at columns to beam/girder connections is critical in developing frame action and must incorporate effects of member shrinkage. Detailed information can be obtained from the national wood research laboratories.

There are many references that describe the engineering practices and procedures used to design wood structures that will perform adequately when subjected to lateral forces. The list at the end of this *Commentary* chapter gives some, but by no means all, of these.

12.3.2 Shear Resistance Based on Principles of Mechanics:

Discussion of cyclic test protocol is included in ATC (1995), Dolan (1996), and Rose (1996).

12.3.3 Deformation Compatibility Requirements: The intent of this section is to require the registered design professional to visualize the deformed shape of the structure to ensure that the connections provide the necessary ductility to allow the probable deflection demand placed on the structure. Unlike steel or other metal structures, wood is not a ductile material and virtually all of the ductility achieved in the structure is from the metal connections. The planned failure mechanism of wood structures must be through the connections, including the nailing of structural panels, otherwise the failure will be brittle in nature. The philosophy of strong elastic columns and yielding beams cannot be projected from steel to wood structures. To enable a wood structure to deform and dissipate energy during a seismic event, the connections must be the weak link in the structure and be ductile. Recent earthquakes, such as that in Northridge, California, have shown failures due to the fact that consideration of deformation compatibility was neglected.

As an example of a compatibility issue, consider the deformation compatibility between a tie-down connector to the tie-down post and the edge nailing of shear wall sheathing to the tie-down post and adjacent bottom plate. Recent testing and observations from the Northridge earthquake have suggested that the tie-down post experiences notable displacement before significant load can be carried through the tie-down connector. This is due, among other things, to the oversizing of the bolt holes in the tie-down post and the deformation and rotation of the tie-down bracket. Anchor bolts connecting the bottom plate to the foundation below tend to attempt to carry the shear wall uplift as the tie-down post moves. The sheathing, however, is nailed to both the bottom plate, which is held in place, and the tie-down post, which is being pulled up. The result is a large deformation demand being placed on the nails connecting the sheathing to the framing. This often results in the nails pulling out of the sheathing at the tie-down post corner and sometimes results in an unzipping effect where a significant portion of the remaining sheathing nailing fails as high loads cause one nailed connection to fail and move on to overstress the next nail. The most effective solution currently known is to limit the slip and deformation at the tie-down post by using a very stiff nailed or screwed tie-down.

Because this is an area where understanding of compatibility issues is just starting to develop, the Sec. 12.3.3 provision uses the wording "shall be considered in design" in lieu of the originally proposed "provision shall be made to ensure..." The intent is to provide guidance while not requiring the impossible. Equations for estimating diaphragm and shear wall deflections are discussed in Sec. 12.4.1 of this commentary.

If necessary, the stiffness of the wood diaphragms and shear walls can be increased with the use of adhesives (if adhesives are to be used, see *Commentary* Sec. 12.4). However, it should be noted that there are no rational methods for determining deflections in diaphragms that are constructed with non-wood sheathing materials. If the nail stiffness values or shear stiffness of non-wood sheathing materials is determined in a scientific manner, such as through experimental cyclic testing (e.g., see Sec. 12.4 of the *Commentary*), the calculations for determining the stiffness of shear panels will be considered validated.

12.3.4 Framing Requirements: All framing that is designed as part of an engineered wood structure must be designed with connectors that are able to transfer the required forces between various *components*. These connectors can be either proprietary hardware or some of the more conventional connections used in wood construction. However, the capacity of these connectors should be designed according to accepted engineering practice to ensure that they will have the capacity to resist the forces. The requirement of columns and posts being framed to full end bearing requires that the force transfer from the column to the base be accomplished through end grain bearing of the wood, not through placing the bolts or other connectors in shear. This requirement is included to ensure adequate capacity for transfer of the vertical forces due to both gravity and overturning moment. Alternatively, the connection can be designed to transfer the full loading through placing the bolts or other connectors in shear neglecting all possible bearing.

The anchorage connections used in engineered wood construction must be capable of resisting the forces that will occur between adjacent members (beams and columns) and elements (diaphragms and shear walls). These connections can utilize proprietary hardware or be designed in accordance with principles of mechanics. Connections are often the cause of structural failures in wood structures, and the registered design professional is cautioned to use conservative values for allowable capacities since most published values are based on monotonic, not cyclic, load applications (National Oceanic and Atmospheric Administration, 1971). Testing has shown that some one-sided bolted connections subject to cyclic loading, such as tie-down devices, do not perform well. This was substantiated by the poor performance of various wood frame elements in structures in the January 1994 Northridge earthquake.

Concrete or masonry wall anchorages using toe nails or nails subject to withdrawal are prohibited by the *Provisions*. It has been shown that these types of connections are inadequate and do not perform well (U.S. Department of Agriculture, National Oceanic and Atmospheric Administration, 1971). Ledgers subjected to cross-grain bending or tension perpendicular to grain also have performed poorly in past earthquakes, and their use is now prohibited by the *Provisions*.

12.3.5 Sheathing Requirements: Sheathing nails should be driven flush with the surface of the panel, and not further. This could result in the nail head creating a small depression in, but not fracturing, the first veneer. This requirement is imposed because of the significant reduction in capacity and ductility observed in shear walls constructed with over-driven nails. It is advised that the edge distance for sheathing nails be increased as much as possible along the bottom of the panel to reduce the potential for the nails to pull through the sheathing.

Unit shear values for structural-use panel sheathing (Sec. 12.4.3.1) have been generally based on tests of shear wall panels with aspect ratios (height to width ratios) of 2/1 to 1/1. Narrower wall segments (i.e. aspect ratios of greater than 2/1) have been a recent concern based on damage observations following the Northridge Earthquake. In response, various limitations on aspect ratios have been proposed. In the *Provisions*, an aspect ratio adjustment, 2w/h, is provided to account for the reduced stiffness of narrow shear wall segments. This adjustment is based on a review of numerous tests of narrow aspect ratio walls by the TS-7 technical subcommittee. The maximum 3.5/1 aspect ratio is recommended based on constructability issues (i.e. placement of hold-downs) as well as reduced stiffness of narrower shear wall segments.

12.3.6 Wood Members Resisting Horizontal Seismic Forces Contributed by Masonry and Concrete: Due to the significant difference in in-plane stiffness between wood and masonry or concrete systems, the use of wood members to resist the seismic forces produced by masonry and concrete is not allowed. This is due to the probable torsional response such a structure will exhibit. There are two exceptions where wood can be considered to be part of the lateral-load-resisting system. The first is when the wood is in the form of a horizontal truss or diaphragm and the lateral loads do not produce rotation of the horizontal member. The second exception is in structures of two stories or less in height. In this case, the capacity of the wood shear walls will be sufficient to resist the lower magnitude loads imposed. Five restrictions are imposed on these structures to ensure hat the structural performance will not include rotational response and the drift will not cause failure of the masonry or concrete portions of the structure.

12.4 DIAPHRAGMS AND SHEAR WALLS: Many wood-framed structures resist seismic forces by acting as a "box system." The forces are transmitted through diaphragms, such as roofs and floors, to reactions provided by shear walls. The forces are, in turn, transmitted to the lower stories and to the final point of resistance, the foundations. A shear wall is a vertical diaphragm generally considered to act as a cantilever from the foundation.

A diaphragm is a nearly horizontal structural unit that acts as a deep beam or girder when flexible in comparison to its supports and as a plate when rigid in comparison to its supports. The analogy to a girder is somewhat more appropriate since girders and diaphragms are made up as assemblies (American Plywood Association, 1991; Applied Technology Council, 1981). Sheathing acts as the "web" to resist the shear in diaphragms and is stiffened by the framing members, which also provide support for gravity loads. Flexure is resisted by the edge elements acting like "flanges" to resist induced tension or compression forces. The “flanges” may be top plates, ledgers, bond beams, or any other continuous element at the perimeter of the diaphragm.

The "flange" (chord) can serve several functions at the same time, providing resistance to loads and forces from different sources. When it functions as the tension or compression flange of the "girder," it is important that the connection to the "web" be designed to accomplish the shear transfer. Since most diaphragm "flanges" consist of many pieces, it is important that the splices be designed to transmit the tension or compression occurring at the location of the splice and to recognize that the direction of application of seismic forces can reverse. It should also be recognized that the shear walls parallel to the flanges may be acting with the flanges to distribute the diaphragm shears. When seismic forces are delivered at right angles to the direction considered previously, the "flange" becomes a part of the reaction system. It may function to transfer the diaphragm shear to the shear wall(s), either directly or as a drag strut between segments of shear walls that are not continuous along the length of the diaphragm.

For shear walls, which may be considered to be deep vertical cantilever beams, the "flanges" are subjected to tension and compression while the "webs" resist the shear. It is important that the "flange" members, splices at intermediate floors, and the connection to the foundation be detailed and sized for the induced forces. In the 1997 Provisions, shear wall aspect ratios, *h/w*, were limited to 2/1 in light of the poor performance of walls with larger aspect ratios in recent tests and in the January 1994 Northridge earthquake, and the results of recent research (Applied Technology Council, 1995; White and Dolan, 1996). In the 2000 *Provisions h/w* up to 3.5/1 are permitted (see sec. 12.3.5).

The "webs" of diaphragms and shear walls often have openings. The transfer of forces around openings can be treated similarly to openings in the webs of steel girders. Members at the edges of openings have forces due to flexure and the higher web shear induced in them and the resultant forces must be transferred into the body of the diaphragm beyond the opening.

In the past, wood sheathed diaphragms have been considered to be flexible by many registered design professionals and model code enforcement agencies. The newer versions of the model codes now recognize that the determination of rigidity or flexibility for determination of how forces will be distributed is dependent on the relative deformations of the horizontal and vertical resisting elements. Wood sheathed diaphragms in structures with wood frame shear walls with various types of sheathing may be relatively rigid compared with the vertical resisting system and, therefore, capable of transmitting torsional lateral forces. A relative deformation of the diaphragm of two or more when compared with the vertical resisting system deformation under the same force is used to define a diaphragm as being flexible.

Discussions of these and other topics related to diaphragm and shear wall design, such as cyclic testing, and pitched or notched diaphragms, may be found in the references.

Deflections: The mid-span deflection of a simple-span blocked structural-use panel diaphragm uniformly nailed throughout may be calculated by use of the following formula:

$$\Delta = \frac{5v\,l^3}{8wEA} + \frac{v\,l}{4Gt} + 0.188\,l\,e_n + \frac{\sum(\Delta_c X)}{2w}$$

where:

Δ	=	the calculated deflection, in millimeters, or inches.
v	=	maximum shear due to factored design loads in the direction under consideration, in kilonewtons per meter, or pounds per lineal foot.
l	=	diaphragm length, in meters, or ft.
w	=	diaphragm width, in meters, or ft.
E	=	elastic modulus of chords, in megapascals, or pounds per square inch.
A	=	area of chord cross-section, in square millimeters, or square inches.
Gt	=	panel rigidity through the thickness, in Newtons per millimeter, or pounds per inch.
e_n	=	nail deformation, in millimeters, or inches
$\Sigma(\Delta_c X)$	=	sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support, in millimeters, or inches.

If not uniformly nailed, the constant 0.188 in the third term must be modified accordingly (See ATC-7, Applied technology Council, 1981).

This formula was developed based on engineering principles and monotonic testing. Therefore, it provides an estimate of diaphragm deflection due to loads applied in the factored resistance shear range. The effects of cyclic loading and resulting energy dissipation may alter the values for nail deformation in the third term as well as chord splice effects of the fourth term, if mechanically-spliced wood chords are used. The formula is not applicable to partially-blocked diaphragms.

The deflection of a blocked structural-use panel shear wall may be calculated by use of the following formula.

$$\Delta = \frac{8v\,h^3}{wEA} + \frac{v\,h}{Gt} + 0.75h\,e_n + \frac{h}{w}\,d_a$$

where:

Δ	=	the calculated deflection, in millimeters, or inches.
v	=	maximum shear due to factored design loads at the top of the wall, in kilonewtons per meter, or pounds per lineal foot.
h	=	shear wall height, in meters, or ft.
w	=	shear wall width, in meters, or ft.
E	=	elastic modulus of boundary element (vertical member at shear wall boundary),in megapascals, or pounds per square inch.
A	=	area of boundary element cross-section (vertical member at shear wall boundary), in square millimeters, or square inches.
Gt	=	panel rigidity through the thickness, in Newtons per millimeter, or pounds per inch.
e_n	=	nail deformation, in millimeters, or inches.
d_a	=	deflection due to anchorage details (rotation and slip at hold downs),in millimeters, or inches.

Guidance for use of the above two equations can be found in References 12-2, 12-3, and 12-4, and ATC-7 (Applied Technology Council, 1981).

The capacity of shear walls shall be determined either from tabulated values that are based on experimental results or from standard principles of mechanics. The tables of allowable values for shear walls sheathed with other than wood or wood-based structural-use panels were eliminated in the 1991 *Provisions* as a result of re-learning the lessons from past earthquakes and testing on the performance of structures sheathed with these materials during the Northridge earthquake. In the 1997 *Provisions* values for capacity for shear walls sheathed with wood-based structural-use panels were reduced from monotonic test values by 10 percent to account for the reduction in capacity observed during cyclic tests. This decision was reviewed for the 2000 edition of the *Provisions* due to the availability of an expanded data set of test results. The reduction was removed for the 2000 *Provisions* when the effect of the test loading protocol was determined to be the cause of the initial perceived reductions. Capacities for diaphragms were not reduced from the monotonic test values because the severe damage that occurred in shear walls has not been noted in diaphragms in recent earthquakes.

One stipulation is that there are no accepted rational methods for calculating deflections for diaphragms and shear walls that are sheathed with materials other than wood-based structural-use panel products fastened with nails. Therefore, if a rational method is to be used, the capacity of the fastener in the sheathing material must be validated by acceptable test procedures employing cyclic forces or displacements. Validation must include correlation between the overall stiffness and capacity predicted by principles of mechanics and that observed from test results. A diaphragm or shear wall sheathed with dissimilar materials on the two faces should be designed as a single-sided wall using the capacity of the stronger of the materials and ignoring the weaker of the materials.

The *Provisions* are based on assemblies having energy dissipation capacities which were recognized in setting the *R* factors. For diaphragms and shear walls utilizing wood framing, the energy dissipation is almost entirely due to nail bending. Fasteners other than nails and staples have not been extensively tested under cyclic load application. When screws or adhesives have been tested in assemblies subjected to cyclic loading, they have had a brittle mode of failure. For this reason, adhesives are prohibited for wood framed shear wall assemblies and only the tabulated values for nailed or stapled sheathing are recommended. Analysis and design of shear wall sheathing applied with adhesives is beyond the scope of the *Provisions*. If one wished to use shear wall sheathing attached with adhesives, as an alternate method of construction in accordance with Sec. 1.2.5, caution should be used (Dolan and White, 1992; Foschi and Filiatrault, 1990). The increased stiffness will result in larger forces being attracted to the structure. The anchorage connections and adjoining assemblies must, therefore, be designed for these increased forces. Due to the brittle failure mode, these walls should be designed to remain elastic, similar to unreinforced masonry. The use of adhesives for attaching sheathing for diaphragms increases their stiffness, and could easily change the diaphragm response from flexible to rigid.

12.4.1 Diaphragms:

12.4.1.1 Horizontal Distribution of Shear: This section of the *Provisions* is intended to define when a diaphragm can be considered to be flexible or rigid. The purpose is to determine whether the diaphragm should have the loads proportioned according to tributary area or stiffness. For flexible diaphragms, the loads should be distributed according to tributary area whereas for rigid diaphragms, the loads should be distributed according to stiffness. The remainder of the intent of this section is covered in the general discussion for Sec. 12.3.4 above.

The distribution of seismic forces to the vertical elements (shear walls) of the lateral force resisting system is dependent, first, on the relative stiffness of the vertical elements versus the horizontal elements and, second, on the relative stiffness of the vertical elements when they have varying deflection characteristics. The first issue is discussed in detail in the *Provisions*, which define when a diaphragm can be considered flexible or rigid and set limits on diaphragms that act in rotation or that cantilever. The second is largely an issue of engineering mechanics, but is discussed in Sec. 12.4 of this commentary because significant variations in engineering practice currently exist.

In situations where a series of vertical elements of the lateral force resisting system are aligned in a row, seismic forces will distribute to the different elements according to their relative stiffness.

Typical current design practice is to distribute seismic forces to a line of structural-use panel sheathed walls in proportion to the lengths of the wall segments such that each segment carries the same unit load. Structural-use panel sheathed wall segments without openings can generally be calculated to have a stiffness in proportion to the wall length when: the tie-down slip is ignored, the structural-use panel sheathing is selected from Tables 12.4.3-2a and b, and the aspect ratio limits of the *Provisions* are satisfied. For stiffness to be proportional to the wall length, the average load per nail for a given nail size must be approximately equal. Conversely, a wall could be stiffened by adding nails and reducing the calculated average load per nail. When including tie-down (hold-down) slip from anchors with negligible slip (1/16 in, 2 mm or less), the assumption of wall stiffness proportional to length is still fairly reasonable. For larger tie-down slip values, wall stiffness will move towards being proportional to the square of the wall length; more importantly, however, the anchorage will start exhibiting displacement compatibility problems as discussed in Sec. 12.3.3. For shear walls with aspect ratios higher than 2/1, the stiffness is no longer in proportion to the length and equations are not available to reasonably calculate the stiffness. For a line of walls with variations in tie-down slip, chord framing, unit load per nail, or other aspects of construction, distribution of load to wall segments will need to be based on a deflection analysis. The shear wall and diaphragm deflection equations that are currently available are not always accurate. As testing results become available, the deflection calculation formulas will need to be updated and design assumptions for distribution of forces reviewed.

Torsional Diaphragm Force Distribution: Sec. 12.4.1.1 defines a diaphragm as being flexible when the maximum lateral deformation of the diaphragm is more than two times the average story drift. Conversely, a diaphragm will be considered rigid when the diaphragm deflection is equal to or less than two times the story drift. This is based on a model building code definition that applies to all materials.

For flexible diaphragms, seismic forces should be distributed to the vertical resisting elements according to tributary area or simple beam analysis. Although rotation of the diaphragm may occur because lines of vertical elements have different stiffness, the diaphragm is not considered stiff enough to redistribute seismic forces through rotation. The diaphragm can be visualized as a single-span beam supported on rigid supports.

For diaphragms defined as rigid, rotational or torsional behavior is expected and results in redistribution of shear to the vertical-force-resisting elements. Requirements for horizontal shear distribution are in Sec. 5.4.4. Torsional response of a structure due to irregular stiffness at any level within the structure can be a potential cause of failure. As a result, dimensional and diaphragm ratio limitations are provided for different categories of rotation. Also, additional requirements apply when the structure is deemed to have a torsional irregularity in accordance with Table 5.2.3.2, Item 1.

In order to understand limits placed on diaphragms acting in rotation, it is helpful to consider two different categories of diaphragms. Category I includes rigid diaphragms that rely on force transfer through rotation to maintain stability. An example would be an open front structure with shear walls on the other three sides. For this more structurally critical category, applicable limitations are:

- Sec. 12.3.6 -- Diaphragm not to be used to resist forces contributed by masonry or concrete in structures over one story.
- Sec. 12.4.1.1, second paragraph -- The length of the diaphragm normal to the opening not to exceed 25 ft (to perpendicular shear walls), and diaphragm *l/w* ratios limited as noted.
- Sec. 12.4.1.1, fourth paragraph -- Additional limitations when rotation is significant enough to be considered a torsional irregularity.

Category II includes rigid diaphragms that have two or more supporting shear walls in each of two perpendicular directions but, because the center of mass and center of rigidity do not coincide, redistribute forces to shear walls through rotation of the diaphragm. These can be further divided into Category IIA where the center of rigidity and mass are separated by a small portion of the structure's least dimension and the magnitude of the rotation is on the order of the accidental rotation discussed in Sec. 5.4.4.2 For this level of rotation, Sec. 12.3.6 Exception 1 might be considered applicable and, as a result, no particular limitations would be placed on diaphragm rotation for Category IIA. Category IIB, rigid diaphragms with eccentricities larger than those discussed in Sec. 5.4.4.2, are subject to the following limitations:

- Sec. 12.3.6 -- Diaphragm not to be used to resist forces contributed by masonry or concrete in structures over one story.
- Sec. 12.4.1.1, fourth paragraph -- Additional limitations when rotation is significant enough to be considered a torsional irregularity.

Sec. 12.4 and Tables 12.4.3-1a and b provide limits for diaphragm ratios. Because flexible diaphragms have very little capacity for distributing torsional forces, further limitation of aspect ratios is used to limit diaphragm deformation such that rigid behavior will occur. The resulting deformation demand on the structure also is limited. Where diaphragm ratios are further limited, exceptions permit higher ratios where calculations demonstrate that higher diaphragm deflections can be tolerated. In this case, it is important to determine the effect of diaphragm rigidity on the horizontal distribution and also the ability of other structural elements to withstand resulting deformations.

Proposals to prohibit wood diaphragms acting in rotation were advanced following the 1994 Northridge earthquake. To date, however, the understanding is that the notable collapses in the Northridge Earthquake occurred in part because of lack of deformation compatibility between the various vertical resisting elements rather than because of the inability of the diaphragm to act in rotation.

Diaphragm Cantilever: Limitations concerning diaphragms that cantilever horizontally past the outermost shear wall (or other vertical element) are related to but distinct from those imposed because of diaphragm rotation. Such diaphragms can be flexible or rigid and for rigid diaphragms can be Category I, IIA or IIB. Both the limitations based on diaphragm rotation (if applicable) and the following limit on diaphragm cantilever must be considered:

- Sec. 12.4.1.1, third paragraph -- Diaphragm cantilever not to exceed the lesser of 25 ft or two thirds of the diaphragm width.

Relative Stiffness of Vertical Elements: In situations where a series of vertical elements of the lateral force resisting system are aligned in a row, the forces will distribute to the different elements according to their relative stiffnesses. This behavior needs to be taken into account whether it involves a series of structural-use panel shear walls of different lengths, a mixture of structural-use panel shear walls with diagonal lumber or non-wood sheathed shear walls, or a mixture of wood shear walls with walls of some other material such as concrete or masonry. See the *Commentary* Sec. 12.3.3 for a discussion of deflection compatibility of structural elements.

12.4.1.2 Aspect Ratio: The *l/w* for a diaphragm and *h/w* for a shear wall discussed in the notations section are intended to be the typical definitions for aspect ratio. The diaphragm span, *l*, is measured perpendicular to the direction of applied force, either for the full dimension of the diaphragm or between supports as appropriate. The width, *w*, is parallel to the applied force (see Figure C12.4.1-1). The *h* of the shear wall is the clear story height (see Figure C12.4.1-2). The alternate definition of aspect ratio is only to be used where specific design and detailing is provided for force transfer around the openings. It is required that the individual wall piers meet the aspect ratio requirement (see Figure C12.4.1-3) and that the overall perforated wall also meet the aspect ratio requirement. Use of the alternate definition involves the design and detailing of chord and collector elements around the opening, and often results in the addition of blocking, strapping and special nailing. As noted, the design for force transfer around the opening must use a rational analysis, and in accordance with ASCE 16 which discusses design principles for shear walls, diaphragms and boundary elements.

12.4.1.4 and 12.4.1.5 Single and Double Diagonally Sheathed Lumber Diaphragms: Diagonally sheathed lumber diaphragms and shear walls are presented in the *Provisions* because they are still used for new construction in some regions. The 1994 *Provisions* contain allowable stress design values. The design values in the 2000 *Provisions* are expressed in terms of the factored shear resistance ($\lambda\phi D$) in order to provide consistency with the tables for wood structural panels. The factored shear resistance is based on a soft conversion from the model code allowable stress loads and capacities to *Provisions* strength loads for regions with high effective peak accelerations. This will allow users in the western states, were this construction is currently being used, to continue with little or no change in requirements; at the same time, reasonable values are provided for regions with lower effective peak accelerations.

12.4.2 Shear Walls:

12.4.2.3 Aspect Ratio: The *l/w* for a diaphragm and *h/w* for a shear wall discussed in the notations section are intended to be the typical definitions for aspect ratio. The diaphragm span, *l*, is measured perpendicular to the direction of applied force, either for the full dimension of the diaphragm or between supports as appropriate. The width, *w*, is parallel to the applied force (see Figure C12.4.1-1). The *h* of the shear wall is the clear story height (see Figure C12.4.1-2). The alternate definition of aspect ratio is only to be used where specific design and detailing is provided for force transfer around the openings. It is required that the individual wall piers meet the aspect ratio requirement (see Figure C12.4.1-3) and that the overall perforated wall also meet the aspect ratio requirement. Use of the alternate definition involves the design and detailing of chord and collector elements around the opening, and often results in the addition of blocking, strapping and special nailing. As noted, the design for force transfer around the opening must use a rational analysis, and in accordance with ASCE 16 which discusses design principles for shear walls, diaphragms and boundary elements.

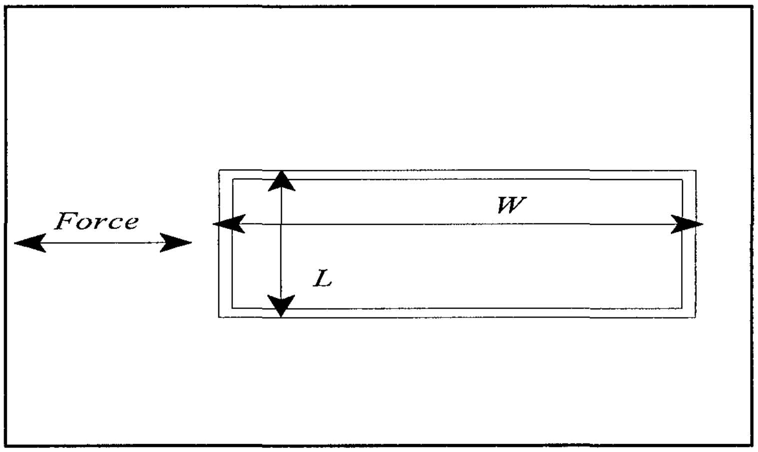


Figure C12.4.1-1. Diaphragm dimension definitions.

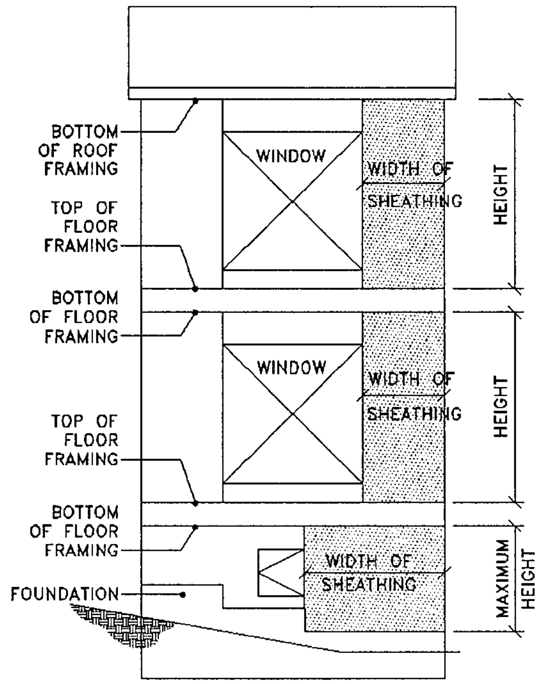


FIGURE 12.4.1-2 Typical shear wall height-to-width ratio.

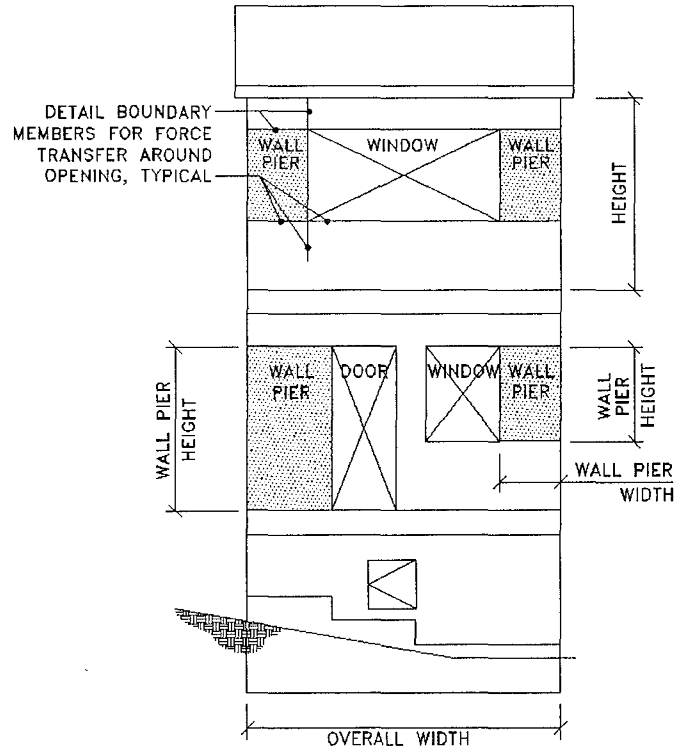


FIGURE C12.4.1-3 Alternate shear wall height-to-width ratio with design for force transfer around openings.

12.4.2.4 Shear Wall Anchorage: Tie-down devices should be based on cyclic tests of the connection to provide displacement capacity that allows rotation of the end post without significant reduction in the shear wall resistance. The strength of the tie-down device should be stronger than the lateral capacity of the wall so that the mechanism of failure is the sheathing fasteners and not a relatively brittle failure of the wall anchorage. For devices for which the published resistance is in allowable stress design values, the nominal strength shall be determined by multiplying the allowable design load by 1.3. The Nominal Strength of a tie-down device may be determined as the average maximum test load resisted without failing under cyclic loading. Average should be based on tests of at least three specimens.

Calculations of deflection of shear walls should include the effects of crushing under the compression chord, uplift of the tension chord, slip in the tie-down anchor with respect to the post, and shrinkage effects of the platforms, which primarily consist of floor framing members. Movement associated with these variables can be significant and neglecting their contribution to the lateral displacement of the wall will results in a significant under-estimation of the deflection.

Custom tie-down devices are permitted to be designed using methods for the particular materials used and ASCE/AF&PA-16 under alternative means and methods.

Tie-down devices that permit significant vertical movement between the tie-down and the tie-down post can cause failure in the nails connecting the shear wall sheathing to the sill plate. High tension and tie-down rotation due to eccentricity can cause the bolts connecting the tie-down bracket to the tie-down post to pull through and split the tie-down post. Devices that permit such movement include heavily loaded one-sided bolted connections with small dimensions between elements resisting rotation due to eccentricity. Any device that uses over-drilled holes such as most bolted connections will also allow significant slip to occur between the device and the tie-down post before load is restrained. Both the NDS and the steel manual specify that bolt holes will be over-drilled as much as 1/16 in (2 mm). This slip is what causes much of the damage to the nails connecting the sheathing to the sill plate. Friction between the tie-down post and the device cannot be counted on to resist load because relaxation in the wood will cause a loss of clamping and, therefore, a loss in friction over time. This is why all tests should be conducted with the bolts "finger tight" as opposed to tightening with a wrench.

Cyclic tests of tie-down connections shall follow a pattern similar to the sequential phased displacement (SPD) tests used by Dolan (1996) and Rose (1996). These tests used full wall assemblies and therefore induced deflection patterns similar to those expected during an earthquake. If full wall assembly tests are not used to test the tie-down devices, it must be shown that the expected rotation as well as tension and compression are used. This is to ensure that walls using the devices will be able to deform in the intended manner. This allows the registered design professional to consider compatibility of deformations when designing the structure.

Splitting of the bottom plate of the shear walls has been observed in tests as well as in structures subjected to earthquakes. Splitting of plates remote from the end of the shear wall can be caused by the rotation of individual sheathing panels inducing upward forces in the nails at one end of the panel and downward forces at the other. With the upward forces on the nails and a significant distance perpendicular to the wall to the downward force produced by the anchor bolt, high cross-grain bending stresses occur. Splitting can be reduced or eliminated by use of large plate washers sufficiently stiff to reduce the eccentricity and by using thicker sill plates. Thicker sill plates (3 in. nominal, 65 mm) are required for all shear walls for which Tables 12.4.3-2a and b require 3 in. nominal (65 mm) framing to prevent splitting due to close nail spacing. This is to help prevent failure of the sill plate due to high lateral loading and cross-grain bending.

The tendency for the nut on a tie-down bracket anchor bolt to loosen significantly during cycled loading has been observed in some testing. One tested method of limiting the loosening is to apply adhesive between the nut and tie-down bolt.

A logical load path for the structure must be provided so that the forces induced in the upper portions of the structure are transmitted adequately through the lower portions of the structure to the foundation.

12.4.2.7, and 12.4.2.8 Single and Double Diagonally Sheathed Lumber Shear Walls: Diagonally sheathed lumber diaphragms and shear walls are presented in the *Provisions* because they are still used for new construction in some regions. The 1994 *Provisions* contain allowable stress design values. The design values in the 2000 *Provisions* are expressed in terms of the factored shear resistance ($\lambda\phi D$) in order to provide consistency with the tables for wood structural panels. The factored shear resistance is based on a soft conversion from the model code allowable stress loads and capacities to *Provisions* strength loads for regions with high effective peak accelerations. This will allow users in the western states, were this construction is currently being used, to continue with little or no change in requirements; at the same time, reasonable values are provided for regions with lower effective peak accelerations.

12.4.3 Perforated Shear Walls: Requirements for the design of perforated shear walls are new to the 2000 *NEHRP Recommended Provisions.*

In a traditional engineering approach for design of shear walls with openings, design force transfer around the openings involves developing a system of piers and coupling beams within the shear wall. Load paths for the shear and flexure developed in the piers and coupling beams generally require blocking and strapping extending from each corner of the opening to some distance beyond. This approach often results in shear wall detailing that is not practical to construct.

The perforated shear wall approach presented in this section utilizes empirically based reductions of wood structural panel shear wall capacities to account for the presence of openings that have not been specifically designed and detailed for moment resistance. This method accounts for the capacity that is inherent in standard construction, rather than relying on special construction requirements. It is not expected that sheathed wall areas above and below openings behave as coupling beams acting end to end, but rather that they provide local restraint at their ends. As a consequence significantly reduced capacities are attributed to interior perforated shear wall segments with limited overturning restraint.

In addition to meeting the general requirements for wood structural panel shear walls, perforated shear walls are required to meet the limitations of Sec. 12.4.3.2, the resistance requirements of Sec. 12.4.3.3, and the anchorage and load path requirements of Sec. 12.4.3.4. Example 1 and Example 2 provide guidance on application of provisions of the perforated shear wall approach.

12.4.3.1 Definitions: The definition of perforated shear wall segment references shear wall aspect ratios.

The $2w/h$ adjustment for calculation of unadjusted factored shear resistance only applies when shear wall segments with w/h greater than 2:1 but not exceeding 3.5:1 are used in calculating perforated shear wall resistance. When shear wall segments with w/h greater than 2:1 are present in a perforated shear wall, but not utilized in calculation of perforated shear resistance wall resistance, calculation of unadjusted factored shear resistance should not include the $2w/h$ adjustment. In many cases, due to the conservatism of the $2w/h$ adjustment, it is advantageous to simply ignore the presence of shear wall segments with w/h greater than 2:1 when calculating perforated shear resistance.

12.4.3.2 Limitations: Perforated shear wall design provisions are applicable to wood structural panel shear walls having characteristics identified in Sec. 12.4.3.2.

a. Perforated shear wall segments located at each end of the perforated shear wall ensure that a minimum length of full height sheathing at each end of a perforated shear wall based on the aspect ratio limits of Sec 12.4.3.1.

b. A factored shear resistance not to exceed 0.64 klf, based on values provided in Tables 12.4.2-6 a and b, is provided to identify a point beyond which other means of shear wall design are likely to be more practical than provisions of Sec. 12.4.3. Connection requirements associated with unadjusted shear resistance grater than 0.64 klf will likely not be practical as other methods of shear wall design will be more efficient.

c. No out of plane offsets are permitted in a perforated shear wall. While the limit on out of plane offsets is not unique to perforated shear walls, it is intended to clearly indicate that a perforated shear wall shall not have out of plane (horizontal) offsets.

d. Collectors for shear transfer to each perforated shear wall segment provide for continuity between perforated shear wall segments. This is typically achieved through continuity of the wall double top plates or by attachment of perforated shear wall segments to a common load distributing element such as a floor or roof diaphragm.

e. Uniform top of wall and bottom of wall elevations are required for use of empirical based shear adjustment factors in Table 12.4.3-1.

f. Limiting perforated shear wall height to 20 ft addresses practical considerations for use of the method as wall heights greater than 20 ft are uncommon.

The width, L, of a perforated shear wall and widths $L1$, $L2$ and $L3$ of perforated shear wall segments are shown in Figure C12.4.3.2. Note that, in accordance with the limitations of Sec. 12.4.3.2 and anchorage requirements of Sec. 12.4.4.4, perforated shear wall segments and overturning restraint are provided at each end of the perforated shear wall.

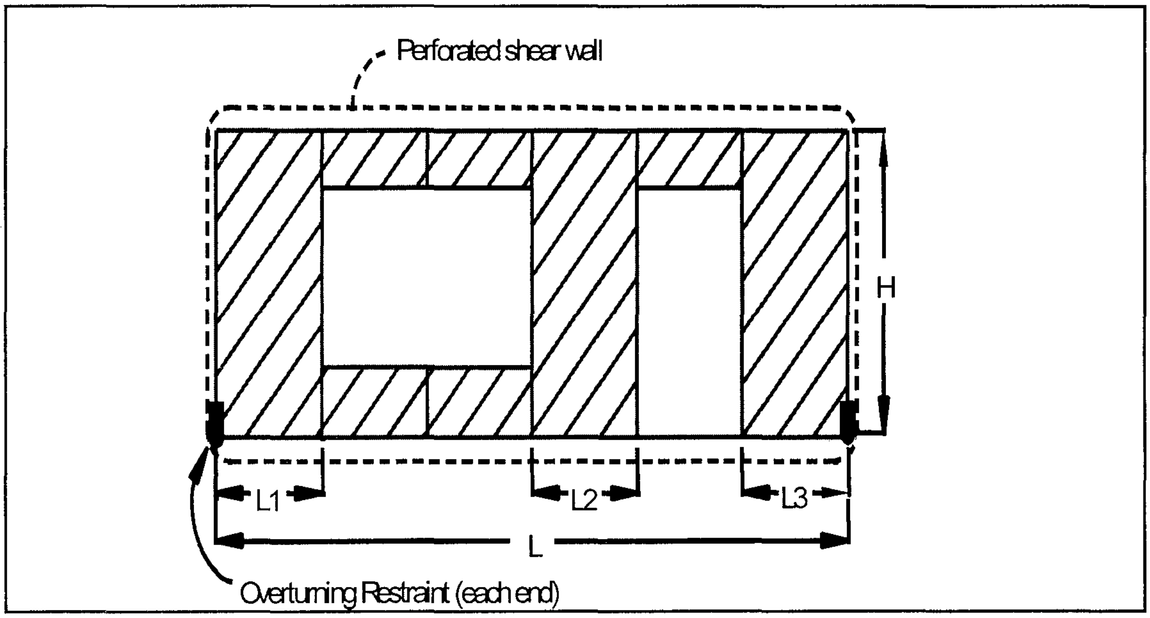


Figure C12.4.3.2 Perforated shear wall.

12.4.3.3 Perforated Shear Wall Resistance: Opening adjustment factors in Table 12.4.2.10.1 are used to reduced shear wall resistance, as provided in Tables 12.4.3-2 a and b for wood structural panel shear walls, based on the percent full-height sheathing and maximum opening height ratio.

Opening adjustment factors in Table 12.4.2.10.1 are based on the following empirical equation for shear capacity ratio, F, which relates the ratio of the shear capacity for a wall with openings to the shear capacity of a fully sheathed wall (Sugiyama, 1981):

$$F = \frac{4}{3 - 2r} \qquad \text{(C12.4.3.3a1)}$$

$$r = \frac{1}{1 + \frac{A_o}{H3L_i}} \qquad \text{(C12.4.3.3b)}$$

where:

r = sheathing area ratio,

A_o = total area of openings,

H = wall height,

ΣL_i = sum of the width of full-height sheathing.

Agreement between Eq. C12.4.3.3a and opening adjustment factors in Table 12.4.3-1 is achieved by recognizing that the tabulated opening adjustment factors are: (1) derived based on an assumption that the height of all openings in a wall are equal to the maximum opening height; and, (2) applied to the sum of the widths of the shear wall segments meeting applicable height to width ratios. The assumption that the height of all openings in a wall are equal to the maximum opening height conservatively simplifies tabular presentation of shear capacity adjustment factors for walls with more than one opening height.

Early verification of Eq. C12.4.3.3a was based on testing of one-third and full-scale shear wall assemblies (Yasumura, 1984; Sugiyama, 1994). More recently, Substantial U.S. verification testing of the influence of openings on shear strength and stiffness has taken place (APA, 1996; Dolan and Johnson, 1996; Dolan and Heine, 1997; NAHB-RC, 1998) indicating shear wall performance is consistent with predictions of Eq. C12.4.3.3a. Results of cyclic testing indicate that the loss in strength due to cyclic loading is reduced for shear walls with openings indicating good relative performance compared to shear walls without openings. Figure 1A provides a graphical summary of some recent U.S. verification testing. Data from monotonic tests of 12 foot shear walls (APA, 1996), monotonic and cyclic tests of long shear walls with unsymmetrically placed openings (Dolan and Johnson, 1996), and monotonic and tests of 16 foot and 20 foot shear walls with narrow wall segments (NAHB-RC, 1998).

Eq. C12.4.3.3a for shear load ratio, F, has been shown to be a good approximation of the stiffness ratio of a wall with openings to that of a fully sheathed wall. Accordingly, the deflection of a perforated shear wall can be calculated as the deflection of an equivalent length fully sheathed wall, divided by the shear load ratio, F. The deflection of a blocked structural-use panel shear wall may be calculated by use of the formula in *Commentary* Sec. 12.4.1.

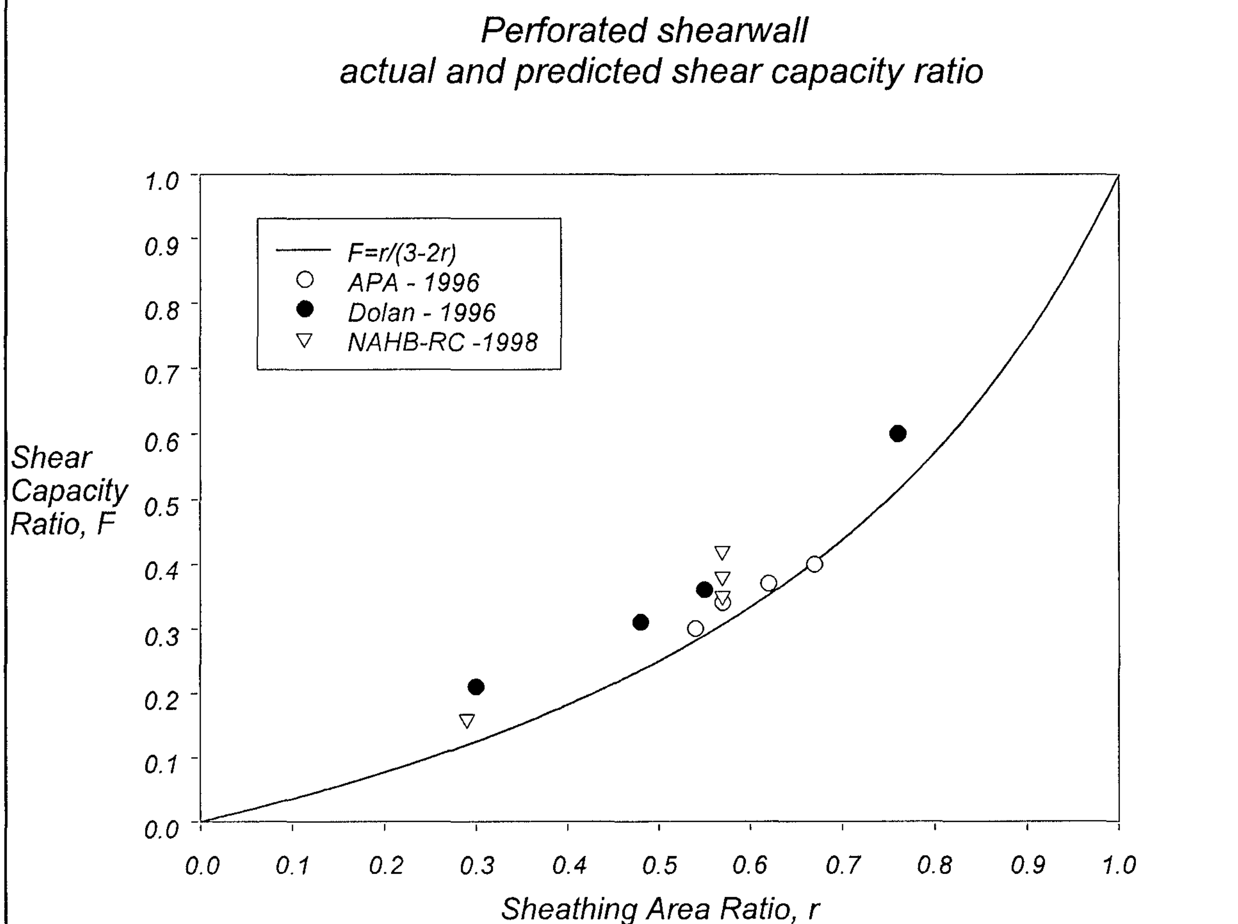


Figure 1A. Shear Capacity Ratio

FIGURE 12.4.3-2 Shear capacity ratio.

Percent full-height sheathing and maximum opening height ratio are used to determine an opening adjustment factor from Table 12.4.3-1. Maximum opening height is the maximum vertical dimension of an opening within the perforated shear wall. A maximum opening height equal to the wall height is used where structural sheathing is not present above or below window openings or above door openings. Percent full-height sheathing is calculated as the sum of the widths of perforated shear wall segments divided by the total length of the shear wall. Sections sheathed full-height which do not meet aspect ratio limits of Sec. 12.4.3.1 for wood structural panel shear walls are not considered in calculation of percent full-height sheathing.

12.4.3.4 Anchorage and Load Path: Anchorage for uplift at perforated shear wall ends, shear, uplift between perforated shear wall ends and compression chord forces are prescribed to address the non-uniform distribution of shear within a perforated shear wall.

Prescribed forces for shear and uplift connections ensure that the capacity of the wall is governed by the sheathing to framing attachment (e.g. shear wall nailing) and not bottom plate attachment for shear and/or uplift. Shear and uplift forces approach the unadjusted factored shear resistance of the perforated shear wall segment as the shear load approaches the shear resistance of the perforated shear wall. A continuous load path to the foundation based on this requirement and consideration of other forces (e.g., from story above) shall be maintained. The magnitude of shear and uplift varies as a function of overturning restraint provided and aspect ratio of the shear wall segment.

12.4.3.4.1 Uplift Anchorage at Perforated Shear Wall Ends: Anchorage for uplift forces due to overturning are required at each end of the perforated shear wall. The required force determined from Eq. 12.4.3.4.1-1 converges on the force required to properly restrain a perforated shear wall segment assuming it develops its unadjusted factored shear resistance. A continuous load path to the foundation based on this requirement and consideration of other forces (e.g., from story above) shall be maintained. In addition, compression chords of perforated shear wall segments are required to transmit compression forces equal to the required tension chord uplift force.

12.4.3.4.2 Anchorage for In-plane Shear: It is required that fastening be provided along the length of the sill plate of wall sections sheathed full-height to resist distributed shear, v, and uplift, t, forces. The resistance required for the shear connection is the average shear over the perforated shear wall segments, divided by the adjustment factor. This resistance will approach the unadjusted factored shear resistance of the wall as the shear wall demand approaches the maximum resistance. This shear fastening resistance will be conservatively accounts for the non-uniform distribution of shear within a perforated shear wall, since it represents the shear that can only be achieved when full overturning restraint is provided.

The provisions of Sec. 12.4.3.2 and Sec. 12.4.3.4.3 requires that this distributed fastening for shear, v, and uplift, t, be provided over the length of full-height sheathed wall sections. With no other specific requirements, the fastening between the full height segments will be controlled by minimum construction fastening requirements. For bottom plates on wood platforms this would only require one 16-penny nail at 16 inches on center. In some cases, it may be preferable to extend a single bottom plate fastening schedule across the entire length of the perforated shear wall rather than require multiple fastening schedules.

12.4.3.4.3 Uplift Anchorage Between Perforated Shear Wall Ends: The resistance required for distributed uplift anchorage, t, is the same as the required shear resistance, v. The adequacy of t can be demonstrated using principles of mechanics and recent testing that determined the capacity of shear wall segments without uplift anchorage. A four foot wide shear wall segment with distributed anchorage of the base plate in lieu of an uplift anchor device provided about 25 percent of the resistance of a segment with uplift anchorage. An eight-foot wide shear wall segment resisted about 45 percent. When these are combined with the resistance adjustment factors, overturning resistance based on the unadjusted factored shear resistance is adequate for perforated shear wall segments with full height openings on each side. Conceptually the distributed uplift resistance, t, is intended to provide the same resistance that anchor bolts at two feet on center provided for tested assemblies. While in the tested assemblies the bottom plates were fastened down, for design it is equally acceptable to fasten down the studs with a strap or similar device, since the studs will in turn restrain the bottom plate.

12.4.3.4.5 Load Path: A continuous load path to the foundation is required for the uplift resistance, *T*; the compression resistance, *C*; the unit shear resistance, *v*; and the unit uplift resistance, *t*. Consideration of accumulated forces (e.g. from story above) is required. Where shear walls occur at the same location at each floor (stack), accumulation of forces is reasonably straightforward. Where shear walls do not stack, attention will need to be paid to maintaining a load path for tie downs at each end of the perforated shear wall, for compression resistance at each end of each perforated shear wall segment, and for distributed forces *v* and *t* at each perforated shear wall segment. Where ends of shear perforated shear wall segments occur over beams or headers, the beam or header will need to be checked for the vertical tension and compression forces in addition to gravity forces. Where adequate collectors are provided at lower floor shear walls, the total shear wall load need only consider the average shear in the perforated shear wall segments above, and not the average shear divided by the adjustment factor.

Example 1 Perforated Shear Wall

Problem Description: The perforated shear wall illustrated in Figure 12.4.4-1 is sheathed with 15/32" wood structural panel with 10d common nails with 4 inch perimeter spacing. All full-height sheathed sections are 4 ft wide. The window opening is 4 ft high by 8 ft wide. The door opening is 6.67 ft high by 4 ft wide. Sheathing is provided above and below the window and above the door. The wall length and height are 24 ft and 8 ft respectively. Holddowns provide overturning restraint at the ends of the perforated shear wall and anchor bolts are used to restrain the wall against shear and uplift between perforated shear wall ends. Determine the shear resistance adjustment factor for this wall.

Solution: The wall defined in the problem description meets the application criteria outlined for the perforated shear wall design method. Holddowns provide overturning restraint at perforated shear wall ends and anchor bolts provide shear and uplift resistance between perforated shear wall ends. Perforated shear wall height, factored shear resistances for the wood structural panel shear wall, and aspect ratio of full height sheathing at perforated shear wall ends meet requirements of the perforated shear wall method.

The process of determining the shear resistance adjustment factor involves determining percent full-heigh sheathing and maximum opening height ratio. Once these are known, a shear resistance adjustment factor can be determined from Table 12.4.3-2a. From the problem description and Figure 12.4.4-1:

Percent full-height sheathing

$$= \frac{\textit{Sum of perforated shear wall segment widths}, \Sigma L}{\textit{Length of perforated shear wall}, L}$$

$$= \frac{4ft + 4ft + 4ft}{24ft} x\, 100 = 50\%$$

Maximum opening height ratio

$$= \frac{\textit{Maximum opening height}}{\textit{Wall height}, h}$$

$$= \frac{6.67ft}{8ft} = \frac{5}{6}$$

For a maximum opening height ratio of 5/6 (or maximum opening height of 6.67 ft when wall height, h, equals 8 ft) and percent full-height sheathing equal to 50 percent, a shear resistance adjustment factor of $C_O = 0.57$ is obtained from Table 12.4.4-1.

Note that if wood structural panel sheathing were not provided above and below the window or above the door the maximum opening height would equal the wall height, h.

Example 2 Perforated Shear Wall

Problem Description: Figure 2 illustrates one face of a 2 story building with the first and second floor walls designed as perforated shear walls. Window heights are 4 ft and door height is 6.67 ft. A trial design is performed in this example based on applied loads, *V*. For simplification, dead load contribution to overturning and uplift restraint is ignored and the effective width for shear in each perforated shear wall segment is assumed to be the sheathed width. Framing is Douglas fir. After basic perforated shear wall resistance and force requirements are calculated, detailing options to provide for adequate shear, ν, and uplift, t, transfer between perforated shear wall ends are covered. Configuration A considers the condition where a continuous rim joist is present at the second floor. Configuration B considers the case where a continuous rim joist is not provided as when floor framing runs perpendicular to the perforated shear wall with blocking between floor framing members.

Perforated Shear Wall Resistance and Force Requirements

Second Floor Wall: Determine wood structural panel sheathing thickness and fastener schedule needed to resist applied load, $V = 2.250$ kips, from the roof diaphragm such that the shear resistance of the perforated shear wall is greater than the applied force. Also determine anchorage and load path requirements for uplift force at ends, in plane shear, uplift between wall ends, and compression.

$$\textit{Percent full-height sheathing} = \frac{4ft + 4ft}{16ft} x\,100 = 50\%$$

$$\textit{Maximum opening height ratio} = \frac{4ft}{8ft} = \frac{1}{2}$$

$$\textit{Shear resistance adjustment factor, } C_o = 0.80$$

Try 15/32 rated sheathing with 8d common nails (0.131 by 2-1/2 in.) At 6 inch perimeter spacing.

Unadjusted shear resistance, Table 12.4.3-2a = 0.36 klf

Adjusted shear resistance

= (unadjusted shear resistance)(C_o)
= (0.36 klf)(0.80) = 0.288 klf

Perforated shear wall resistance

= (Adjusted Shear Resistance)(Σ L_i)
= (0.288 klf)(4 ft + 4 ft) = 2.304 kips
2.304 kips > 2.250 kips ✔ OK

Required resistance due to story shear forces, *V*:

Overturning at shear wall ends, *T*:

$$T = \frac{Vh}{C_o \sum L_i} = \frac{2.250\,kips\,(8ft)}{0.80\,(4ft + 4ft)} = 2.813\ kips$$

In-plane shear, *ν*:

$$\nu = \frac{V}{C_o \sum L_i} = \frac{2.250\,kips}{0.80\,(4ft + 4ft)} = 0.352\ klf$$

Uplift, *t*, between wall ends:

$t = \nu = 0.352$ klf

Compression chord force, *C*, at each end of each perforated shear wall segment:

$C = T = 2.813$ kips

First Floor Wall: Determine wood structural panel sheathing thickness and fastener schedule needed to resist applied load, $V = 2.600$ kips, at the second floor diaphragm such that the shear resistance of the perforated shear wall is greater than the applied force. Also determine anchorage and load path requirements for uplift force at ends, in plane shear, uplift between wall ends, and compression.

$$\textit{Percent full-height sheathing} = \frac{4ft + 4ft}{12ft} x\,100 = 67\%$$

$$\textit{Shear resistance adjustment factor}, C_o = 0.67$$

Unadjusted shear resistance - Table 12.4.3-2a
= 0.49 klf

Adjusted shear resistance
= (Unadjusted Shear Resistance)(C_o)
= (0.49 klf)(0.67) = 0.328 klf

Perforated shear wall resistance
= (Adjusted Shear Resistance)(ΣL_i)
= (0.328 klf)(4 ft + 4 ft) = 2.626 kips
2.626 kips > 2.600 kips ✔ OK

Required resistance due to story shear forces, *V*:

Overturning at shear wall ends, *T*:

$$T = \frac{V h}{C_o \sum L_i} = \frac{2.600\ kips\ (8\,ft)}{0.67\ (4\,ft + 4\,ft)} = 3.880\ kips$$

When maintaining load path from story above,

T = T from second floor + T from first floor
= 2.813 kips + 3.880 kips = 6.693 kips

In-plane shear, *v*:

$$v = \frac{V}{C_o \sum L_i} = \frac{2.600\ kips}{0.67\ (4\,ft + 4\,ft)} = 0.485\ klf$$

Uplift, t, between wall ends:

$t = v = 0.485$ klf

Uplift, *t*, can be cumulative with 0.352 klf from story above to maintain load path. Whether this occurs depends on detailing for transfer of uplift forces between end walls.

Compression chord force, *C*, at each end of each perforated shear wall segment:

$C = T = 3.880$ kips

When maintaining load path from story above, C = 3.880 kips + 2.813 kips = 6.693 kips.

Holddowns and posts and the ends of perforated shear wall are sized using calculated force, *T*. The compressive force, *C*, is used to size compression chords as columns and ensure adequate bearing.

Configuration A - Continuous Rim Joist (see Figure 3)

***Second Floor*:**
Determine fastener schedule for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.352$ klf.

Wall bottom plate (1 ½" thickness) to rim joist. Use 20d box nail (0.148 by 4 in.). Lateral resistance $\phi\lambda Z' = 0.254$ kips per nail and withdrawal resistance $\phi\lambda W' = 0.155$ kips per nail.

Nails for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.352 klf / 0.254 kips per nail
= 1.39 nails per foot

Nails for uplift transfer
= (uplift force, t)/$\phi\lambda W'$
= 0.352 klf / 0.155 kips per nail
= 2.27 nails per foot

Net spacing for shear and uplift
= 3.3 inches on center

Rim joist to wall top plate. Use 8d box nails (0.113 by 2-1/2 in.) toe-nailed to provide shear transfer. Lateral resistance $\phi\lambda Z' = 0.129$ kips per nail.

Nails for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.352 klf / 0.129 kips per nail
= 2.73 nails per foot

Net spacing for shear
= 4.4 inches on center

See detail in Figure 2 for alternate means a shear transfer (e.g metal angle or plate connector).

Transfer of uplift, *t*, from second floor in this example is accomplished through attachment of second floor wall to the continuous rim joist which has been designed to provide sufficient strength to resist the induced moments and shears. Continuity of load path is provided by holddowns at the ends of the perforated shear wall.

First Floor: Determine anchorage for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.485$ klf.

Wall bottom plate (1 ½" thickness) to concrete. Use ½ inch anchor bolt with lateral resistance $\phi\lambda Z'$ = 1.34 kips.

Bolts for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.485 klf / 1.34 kips per bolt
= 0.36 bolts per foot

Net spacing for shear
= 33 inches on center

Bolts for uplift transfer. Check axial capacity of bolts for $t = v = 0.485$ klf and size plate washers accordingly. No interaction between axial and lateral load on anchor bolt is assumed (e.g. presence of axial tension does not affect lateral strength).

Configuration B - Blocking Between Joists (see Figure 3)

***Second Floor*:**
Determine fastener schedule for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.352$ klf.

Wall bottom plate (1 ½" thickness) to rim joist. Use 20d box nail (0.148 by 4 in.). Lateral resistance $\phi\lambda Z'$ = 0.254 kips per nail.

Nails for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.352 klf / 0.254 kips per nail
= 1.39 nails per foot

Net spacing for shear
= 8.63 inches on center

Rim joist to wall top plate. Use 8d box nails (0.113 by 2-1/2 in.) toe-nailed to provide shear transfer. Lateral resistance $\phi\lambda Z'$ = 0.129 kips per nail.

Nails for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.352 klf / 0.129 kips per nail
= 2.73 nails per foot

Net spacing for shear
= 4.4 inches on center

See detail in Figure 3 for alternate means a shear transfer (e.g metal angle or plate connector).

Stud to stud. Provide a metal strap for transfer of uplift, t, from second story wall studs to first story wall studs. Size strap for 0.352 klf uplift and place at 2 ft on center to coincide with stud spacing. This load path will be maintained by transfer of forces through first floor wall framing to the foundation.

***First Floor*:**
Determine anchorage for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.485$ klf.

Wall bottom plate (1 ½" thickness) to concrete. Use ½ inch anchor bolt with lateral resistance $\phi\lambda Z'$ = 1.34 kips.

Bolts for shear transfer
= (shear force, v)/$\phi\lambda Z'$
= 0.485 klf / 1.34 kips per bolt
= 0.36 bolts per foot

Net spacing for shear
= 33 inches on center

Uplift transfer:

A metal strap embedded in concrete at 2 ft on center and attached to first story studs maintaining load path with second story is used. In this case all uplift forces, t, between perforated shear wall ends are resisted by the metal strap. Size metal strap and provide sufficient embedment for uplift force, t = 0.485 klf + 0.352 klf = 0.837 klf.

An alternative detail for uplift transfer uses a metal strap lapped under bottom plate. Size metal strap, anchor bolt, and plate washers for uplift force, t = 0.485 klf + 0.352 klf = 0.837 klf to maintain load path from the second story. No interaction between axial and lateral load on anchor bolt is assumed (e.g. presence of axial tension does not affect lateral strength).

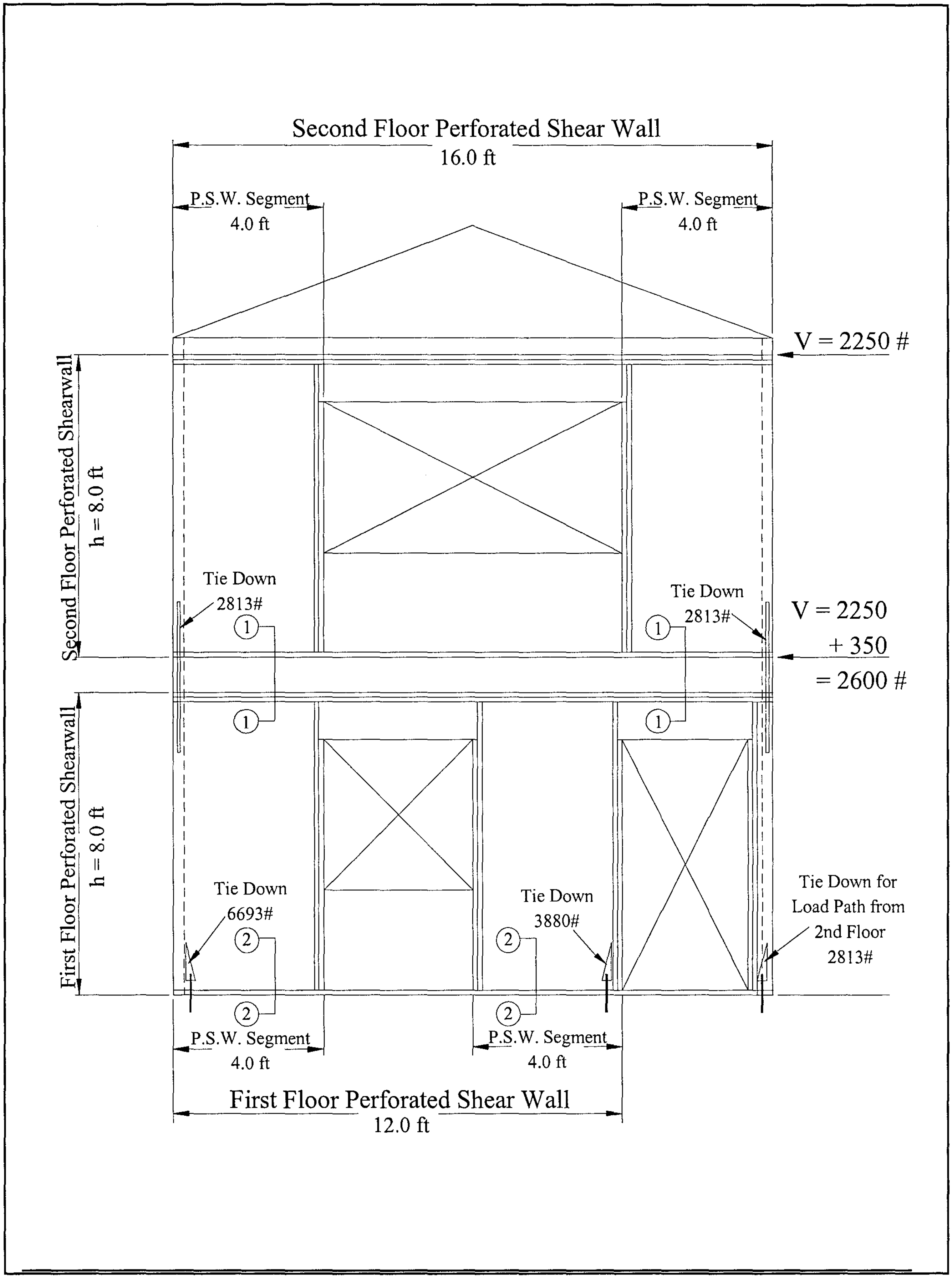
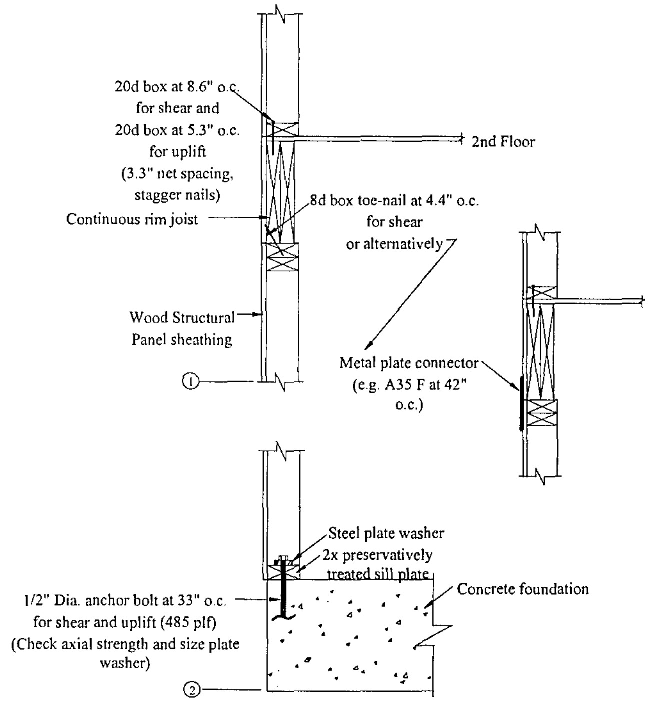
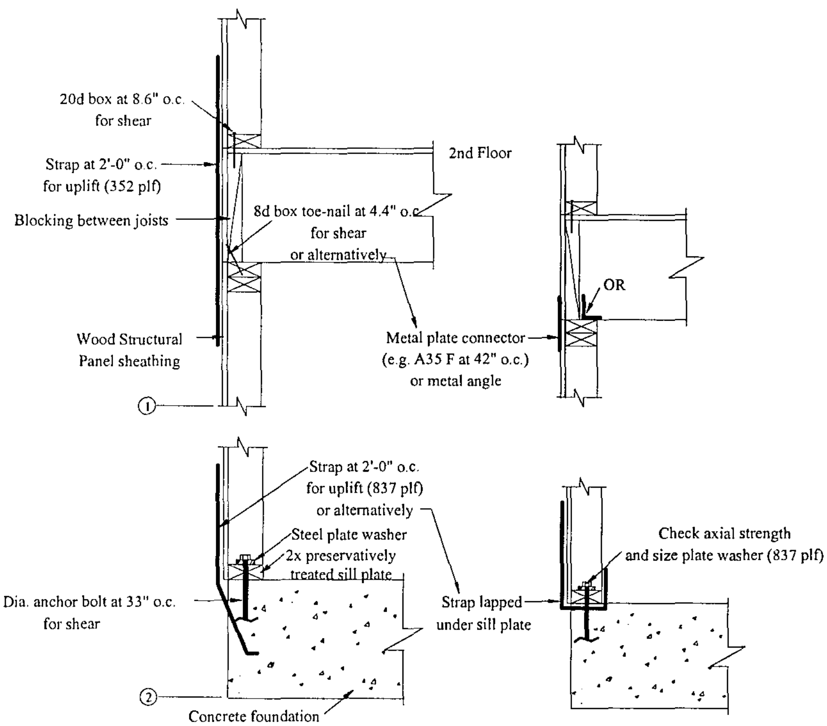

Second Floor Perforated Shear Wall
16.0 ft
P.S.W. Segment
4.0 ft
P.S.W. Segment
4.0 ft
V = 2250 #
Second Floor Perforated Shearwall
h = 8.0 ft
Tie Down
2813#
Tie Down
2813#
V = 2250
+ 350
= 2600 #
1
1
1
1
First Floor Perforated Shearwall
h = 8.0 ft
Tie Down
6693#
Tie Down
3880#
Tie Down for
Load Path from
2nd Floor
2813#
2
2
2
2
P.S.W. Segment
4.0 ft
P.S.W. Segment
4.0 ft
First Floor Perforated Shear Wall
12.0 ft

Figure 3.

Configuration A



Configuration B



12.5 CONVENTIONAL LIGHT-FRAME CONSTRUCTION: The *Provisions* intend that a structure using conventional construction methods and complying with the requirements of this section be deemed capable of resisting the seismic forces imposed by the *Provisions*. Repetitive framing members such as joists, rafters, and studs together with sheathing and finishes comprise conventional light-frame construction. The subject of conventional construction is addressed in each of the model codes. It is acknowledged and accepted that, for the most part, the conventional construction provisions in the model codes concerning framing members and sheathing that carry gravity loads are adequate. This is due to the fact that the tables in the model codes giving allowable spans have been developed using basic principles of mechanics. For seismic lateral force resistance, however, experience has shown that additional requirements are needed.

To provide lateral force resistance in vertical elements of structures, wall bracing requirements have been incorporated in conventional construction provisions of the model codes. With a few exceptions, these generally have been adequate for single family residences for which conventional construction requirements were originally developed. While the model building codes have been quite specific as to the type of bracing materials to be used and the amount of bracing required in any wall, no limits on the number or maximum separation between braced walls have been established. This section of the *Provisions* introduces the concept of mandating the maximum spacing of braced wall lines. By mandating the maximum spacing of braced wall lines and thereby limiting the lateral forces acting on these vertical elements, these revisions provide for a lateral-force-resisting system that will be less prone to overstressing and that can be applied and enforced more uniformly than previous model building code requirements. While specific elements of light-frame construction may be calculated to be overstressed, there is typically a great deal of redundancy and uncounted resistance in such structures and they have generally performed well in past earthquakes. The experience in the Northridge earthquake was, however, less reassuring, especially for those residences relying on gypsum board or stucco for lateral force resistance. The light weight of conventional construction, together with the large energy dissipation capacity of the multiple fasteners used and inherent redundancy of the system are major factors in the observed good performance where wood or wood-based panels were used.

The scope of this section specifically excludes prescriptive design of structures with concrete or masonry walls above the basement story, with the exception of veneer, in order to maintain the light weight of construction that the bracing requirements are based on. Wood braced wall panels and diaphragms as prescribed in this section are not intended to support lateral forces due to masonry or concrete construction. Prescriptive (empirical) design of masonry walls is allowed for in Chapter 11; however, design of structures combining masonry wall construction and wood roof and floor diaphragm construction must have an engineered design. In regions of high seismic activity, past earthquakes have demonstrated significant problems with structures combining masonry and wood construction. While engineered design requirements do address these problems, the prescriptive requirements in the model codes do not adequately address these problems. Masonry and concrete basement walls are permitted to be constructed in accordance with the requirements of CABO Code.

12.5.1.1 Irregular Structures: This section was added to the 1997 *Provisions* to clarify the definition of irregular (unusually shaped) structures that would require the structure to be designed for the forces prescribed in Chapter 5 in accordance with the requirements of Sec. 12.3 and 12.4. The descriptions and diagrams provide the registered design professional with several typical irregularities that produce torsional response, or result in forces considered high enough to require an engineered design and applies only to Seismic Design Category C and D structures.

Structures with geometric discontinuities in the lateral force resisting system have been observed to sustain more earthquake and wind damage than structures without discontinuities. They have also been observed to concentrate damage at the discontinuity location. For Seismic Design Categories C and D, this section translates applicable irregularities from Tables 5.2.3.2 and 5.2.3.3 into limitations on conventional light-frame construction. When a structure falls within the description of irregular, it is required that either the entire structure or the non-conventional portions be engineered in accordance with the engineered design portions of the *Provisions*. The irregularities are based on similar model code requirements. While conceptually these are equally applicable to all Seismic Design Categories, they are more readily accepted in areas of high seismic risk, where damage due to irregularities has repeatedly been observed.

The engineered design of non-conventional portions in lieu of the entire structure is a common practice in some regions. The registered design professional is left to judge the extent of the portion to be designed. This often involves design of the nonconforming element, force transfer into the element, and a load path from the element to the foundation. A nonconforming portion will sometimes have enough of an impact on the behavior of a structure to warrant that the entire lateral-force-resisting system receive an engineered design.

12.5.1.1.1: This limitation is based on Item 4 of Table 5.2.3.3 and applies when braced wall panels are offset out-of-plane from floor to floor. In-plane offsets are discussed in another item. Ideally braced wall panels would always stack above of each other from floor to floor with the length stepping down at upper floors as less length of bracing is required.

Because cantilevers and set backs are very often incorporated into residential construction, the exception offers rules by which limited cantilevers and setbacks can be considered conventional. Floor joists are limited to 2 by 10 (actual 1½ by 9¼ in., 38 by 235 mm) or larger and doubled at braced wall panel ends in order to accommodate the vertical overturning reactions at the end of braced wall panels. In addition the ends of cantilevers are attached to a common rim joist to allow for redistribution of load. For rim joists that cannot run the entire length of the cantilever, the metal tie is intended to transfer vertical shear as well as provide a nominal tension tie. Limitations are placed on gravity loads to be carried by cantilever or setback floor joists so that the joist strength will not be exceeded. The roof loads discussed are based on the use of solid sawn members where allowable spans limit the possible loads. Where engineered framing members such as trusses are used, gravity load capacity of the cantilevered or setback floor joists should be carefully evaluated.

12.5.1.1.2: This limitation is based in Item 1 of Table 5.2.3.2, and applies to open-front structures or portions of structures. The conventional construction bracing concept is based on using braced wall lines to divide a structure up into a series of boxes of limited dimension, with the seismic force to each box being limited by the size. The intent is that each box be supported by braced wall lines on all four sides, limiting the amount of torsion that can occur. The exception, which permits portions of roofs or floors to extend past the braced wall line, is intended to permit construction such as porch roofs and bay windows. Walls with no lateral resistance are allowed in areas where braced walls are prohibited.

12.5.1.1.3: This limitation is based on Item 4 of Table 5.2.3.3 and applies when braced wall panels are offset in-plane. Ends of braced wall panels supported on window or door headers can be calculated to transfer large vertical reactions to headers that may not be of adequate size to resist these reactions. The exception permits a 1 foot extension of the braced wall panel over a 4 by 12 (actual 3½ by 11¼ in., 89 by 286 mm) header on the basis that the vertical reaction is within a 45 degree line of the header support and therefore will not result in critical shear or flexure. All other header conditions require an engineered design. Walls with no lateral resistance are allowed in areas where braced walls are prohibited.

12.5.1.1.4: This limitation results from observation of damage that is somewhat unique to split-level wood frame construction. If floors on either side of an offset move in opposite directions due to earthquake or wind loading, the short bearing wall in the middle becomes unstable and vertical support for the upper joists can be lost, resulting in a collapse. If the vertical offset is limited to a dimension equal to or less than the joist depth, then a simple strap tie directly connecting joists on different levels can be provided, and the irregularity eliminated. CABO *One- and Two-Family Dwelling Code* Sec. 502.4.1 provides requirements for tying of floor joists.

12.5.1.1.5: This limitation is based on Item 5 of Table 5.2.3.3 and applies to nonperpendicular braced wall lines. When braced wall lines are not perpendicular to each other, further evaluation is needed to determine force distributions and required bracing.

12.5.1.1.6: This limitation is based on Item 3 of Table 5.2.3.2 and attempts to place a practical limit on openings in floors and roofs. Because stair openings are essential to residential construction and have long been used without any report of life-safety hazards resulting, these are felt to be acceptable conventional construction. See Sec. 12.5.3.7 for detailing requirements for permitted openings.

12.5.1.1.7: This limits a condition that can cause a torsional irregularity per Item 1 of Table 5.2.3.2. Where heights of braced wall panels vary significantly, distribution of lateral forces will also vary. If a structure on a hill is supported on 2 foot high braced cripple wall panels on one side and 8 foot high panels on the other, torsion and redistribution of forces will occur. An engineered design for this situation is required in order to evaluate force distribution and provide adequate wall bracing and anchor bolting. This limitation applies specifically to walls from the foundation to the floor. While gable-end walls have similar variations in wall heights, this has not been observed to be a significant concern in conventional construction. See Sec. 12.5.3.6 for detailing requirements for permitted foundation stepping.

12.5.2 Braced Walls:

12.5.2.1 Spacing Between Braced Wall Lines: Table 12.5.1-1 prescribes the spacing of braced wall lines and number of stories permitted for conventional construction structures. Figures C12.5.2.1-1 and C12.5.2.1-2 illustrate the basic components of the lateral bracing system. Information in Tables 12.5.1-1 and 12.5.2-1 was first included in the 1991 Edition.

12.5.2.2 Braced Wall Line Sheathing Requirements: Table 12.5.2-1 prescribes the minimum length of bracing along each 25 ft (7.6 m) length of braced wall line. (See *Commentary* Sec. 12.4 regarding adhesive attachment.) Total height of structures has been reduced to limit overturning of the braced walls so that significant uplift is not generally encountered. The height limit will accommodate 8 to 10 ft (2.4 to 3 m) story heights.

12.5.3 Detailing Requirements: The intent of this section is to rely on the traditional light-frame conventional construction materials and fastenings as prescribed in the references for this chapter. Braced wall panels are not required to be aligned vertically or horizontally (within the limits prescribed in Sec. 12.5.1.1) but stacking is desirable where possible. With the freedom provided for non-alignment it becomes important that a load path be provided to transfer lateral forces from upper levels through intermediate vertical and horizontal resisting elements to the foundation. Connections between horizontal and vertical resisting elements are prescribed. In structures two or three stories in height, it is desirable to have interior braced wall panels supported on a continuous foundation. See Figures C12.5.3-1 through C12.5.3-11 for examples of connections.

The 1997 *Provisions* incorporates some of the wall anchorage, top plate, and braced wall panel connection requirements from the model building codes. These are included for completeness of the document and to clarify the requirement for the registered design professional. Additional requirements for foundations supporting braced wall panels has also been added to provide guidance and clarity for the registered design professional.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C,D AND E	25
B	35
A	NOT REQUIRED

*** REFER TO TABLE 12 . 5. 2-1 FOR MINIMUM LENGTH OF WALL BRACING**

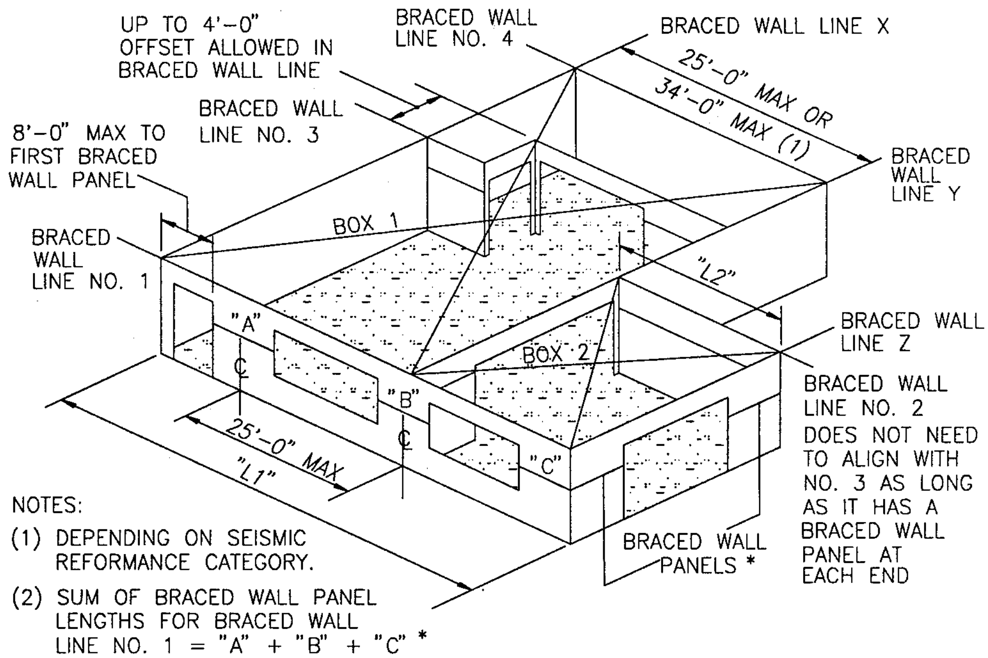


FIGURE C12.5.2.1-1 Acceptable one-story bracing example.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C AND D	25
B	35
A	NOT REQUIRED

*** REFER TO TABLE 12 . 5 . 2 - 1 FOR MINIMUM LENGTH OF WALL BRACING**

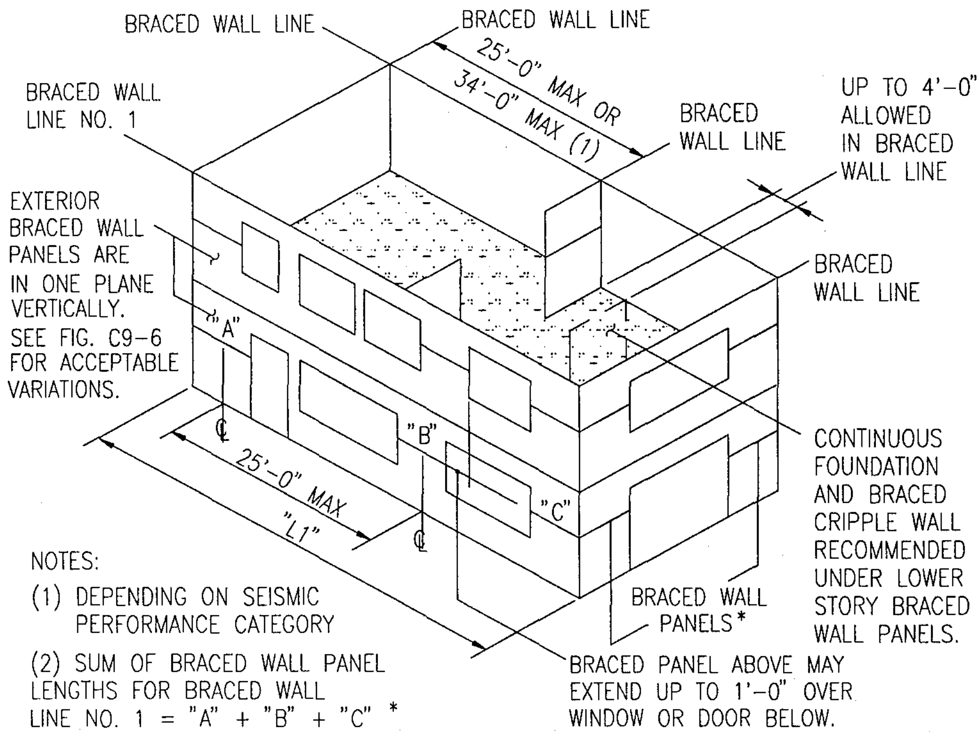
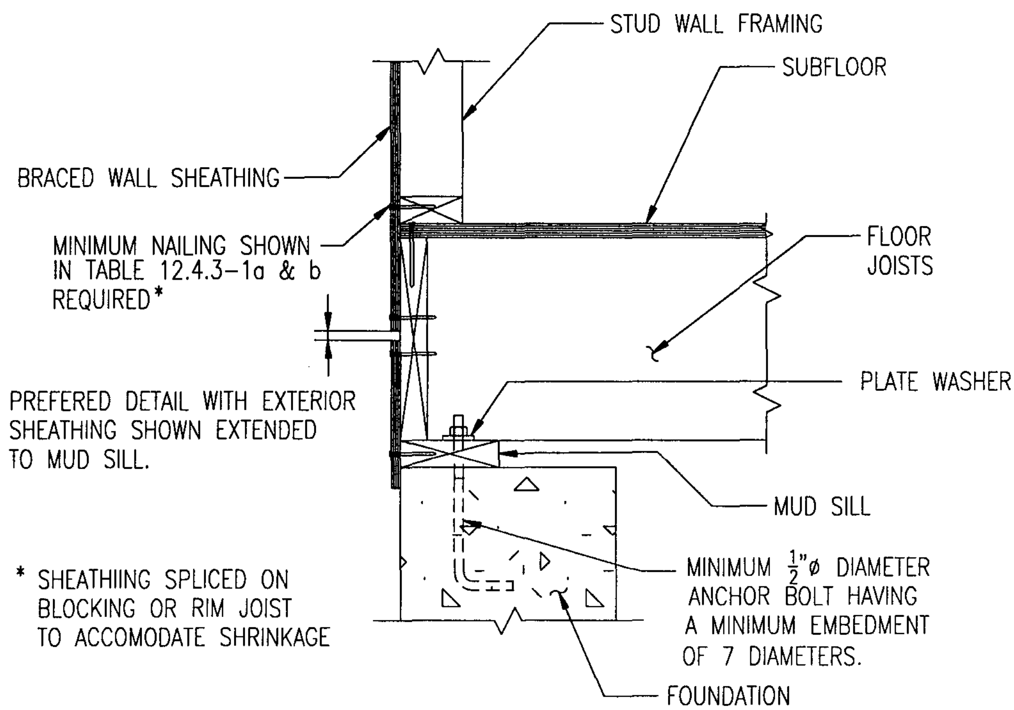


FIGURE C12.5.2.1-2 Acceptable two-story bracing example.



NUMBER OF STORIES	SPACING OF ANCHOR BOLTS (FEET)
ONE OR TWO	6
OVER TWO	4

FIGURE C12.5.3-1 Wall anchor detail.

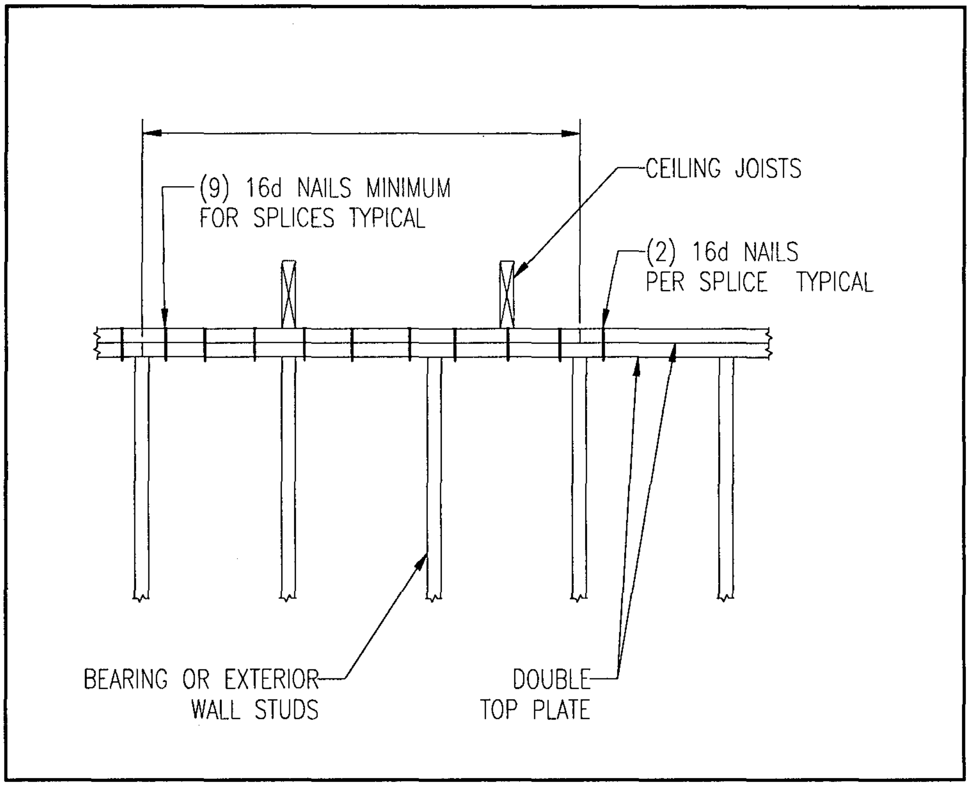


FIGURE C12.5.3-2 Double top plate splice.

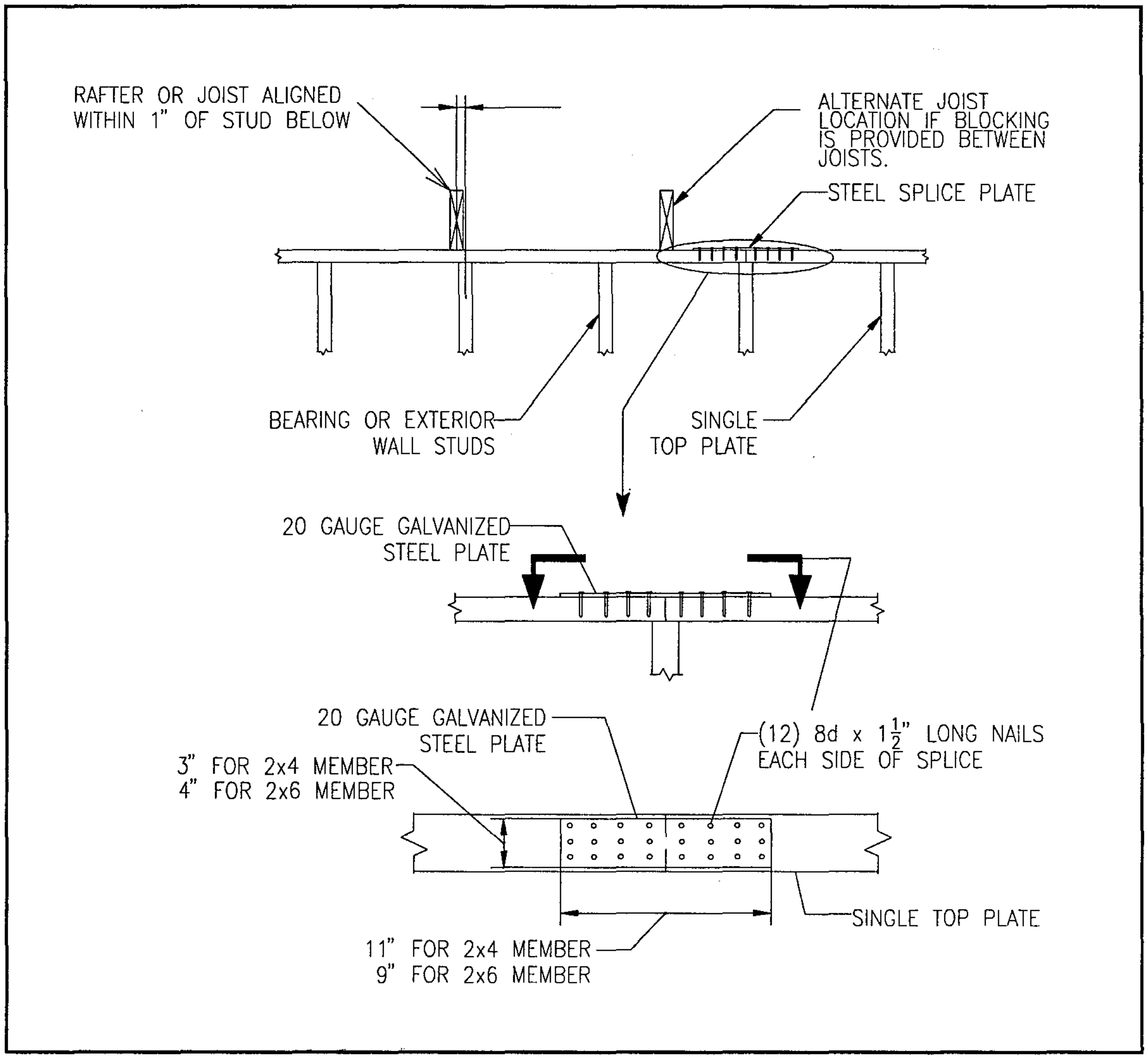


FIGURE C12.5.3-3 Single top plate splice.

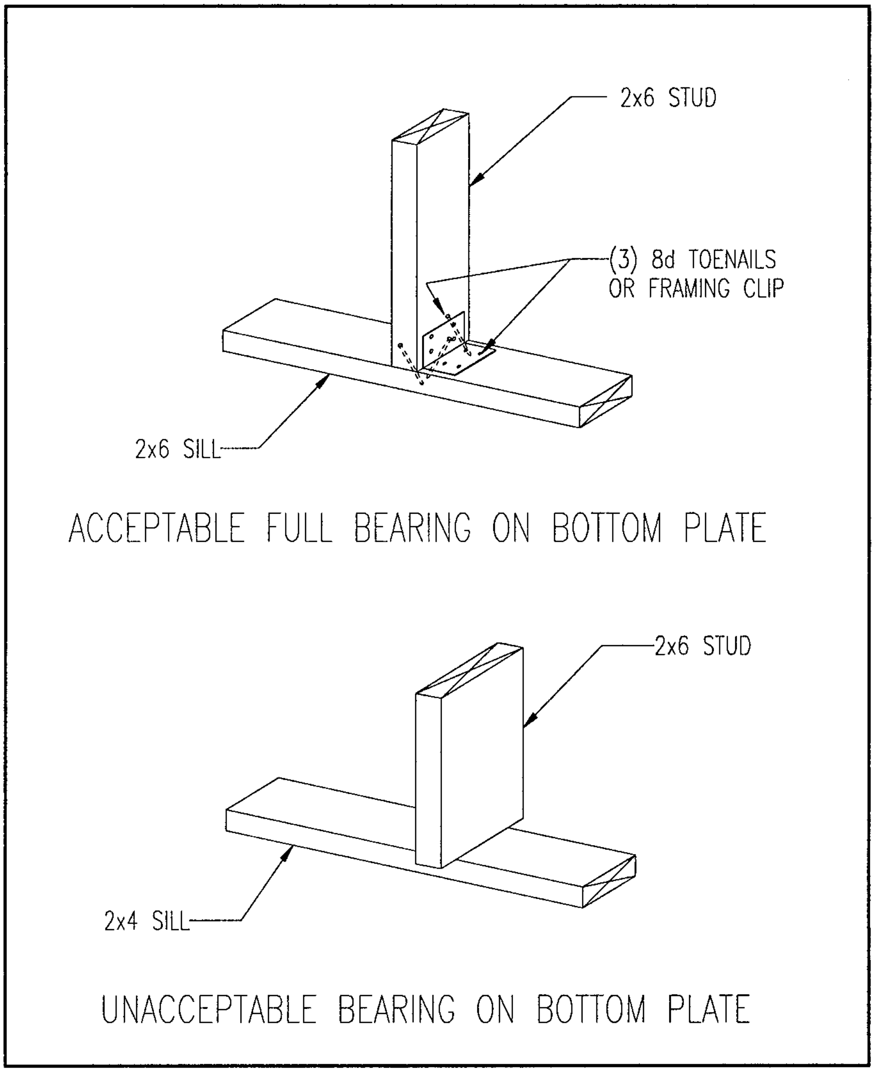


FIGURE C12.5.3-4 Full bearing on bottom plate.

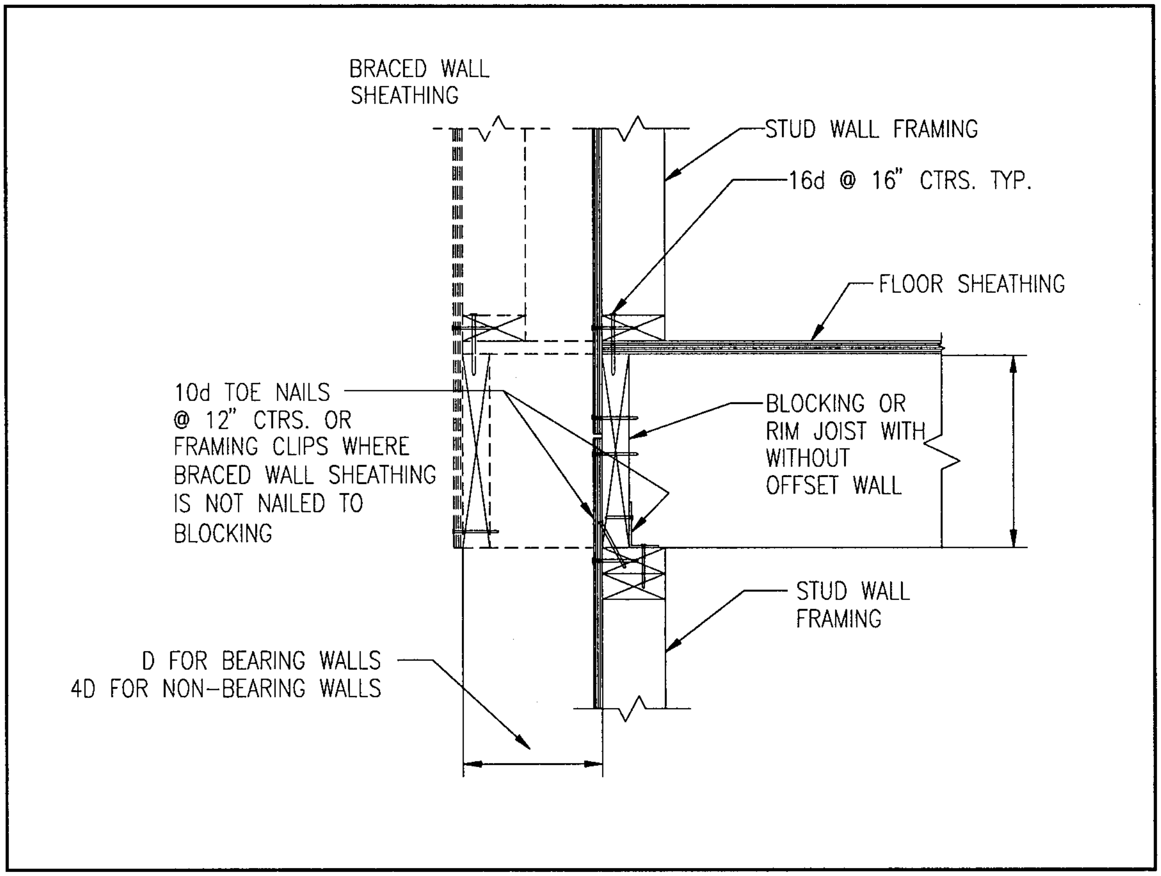


FIGURE C12.5.3-5 Exterior braced wall.

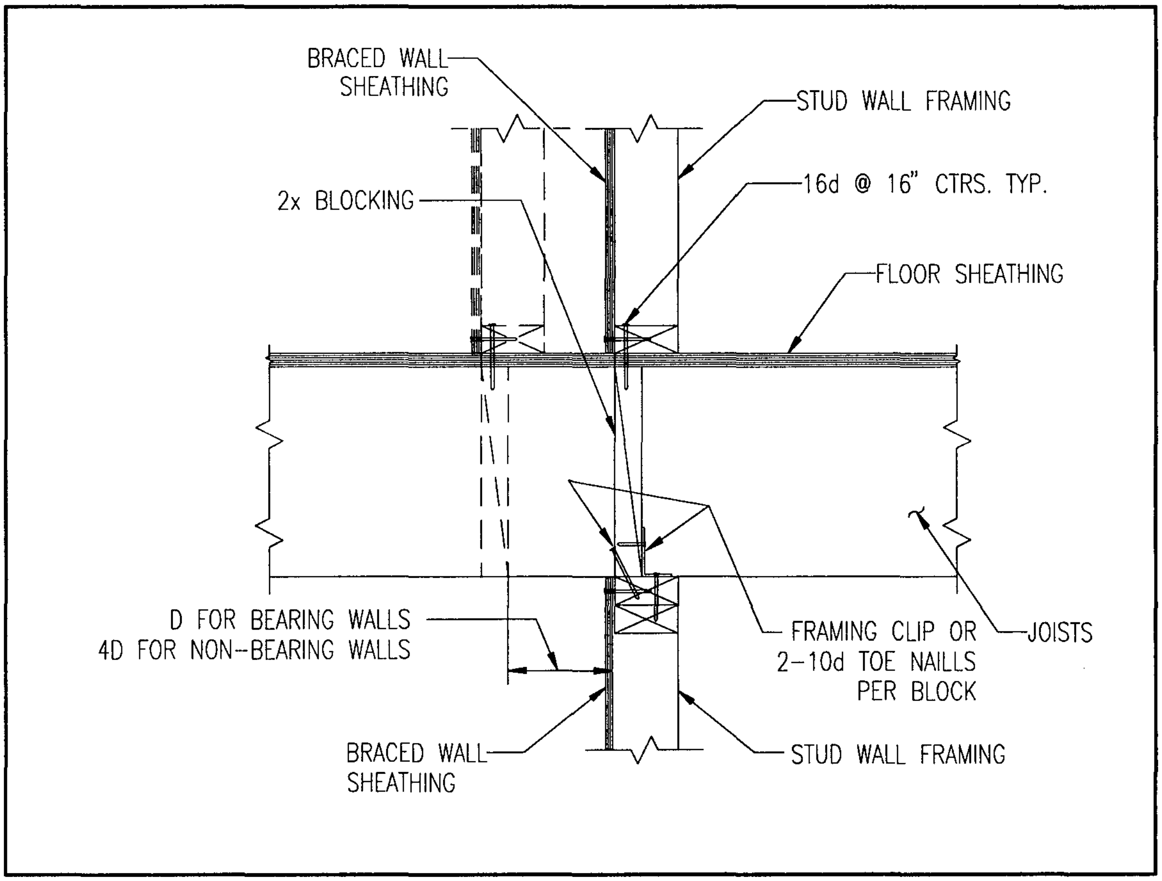


FIGURE C12.5.3-6 Interior braced wall at perpendicular joist.

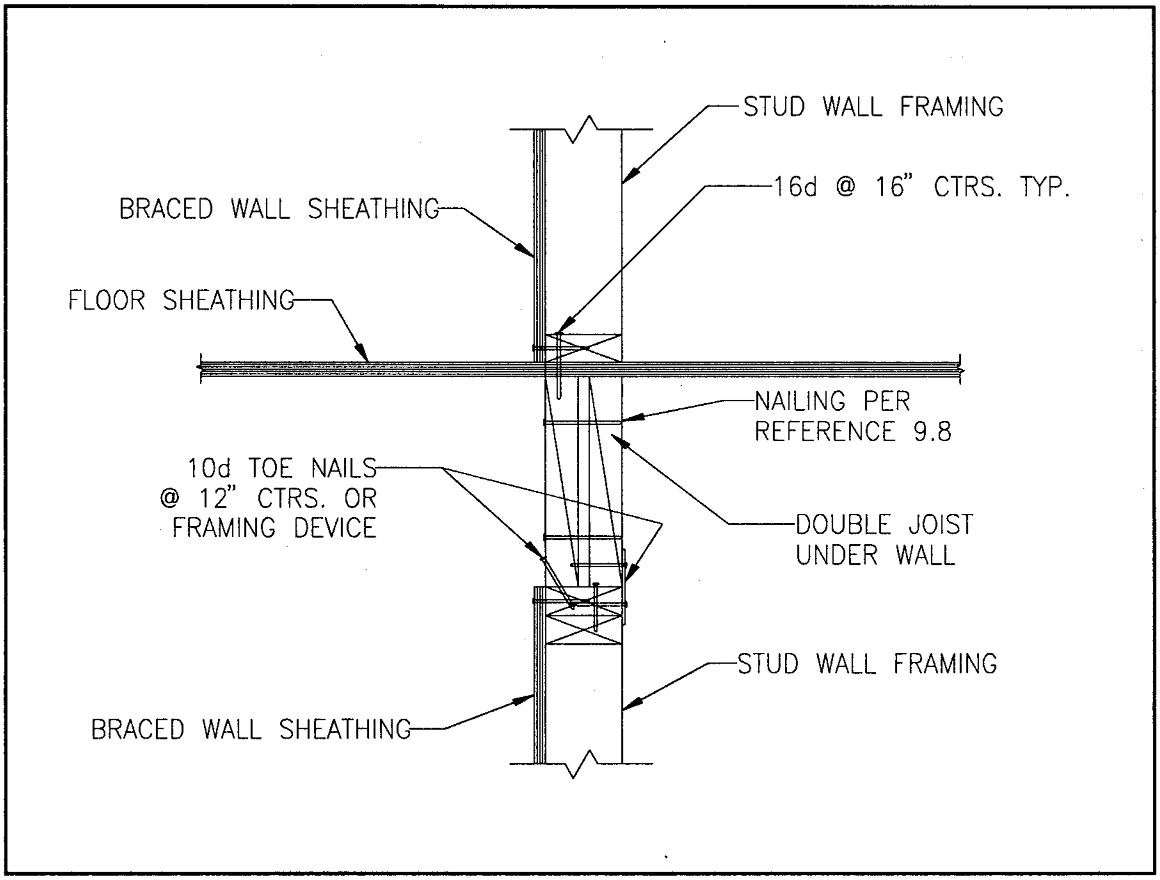


FIGURE C12.5.3-7 Interior braced wall at parallel joist.

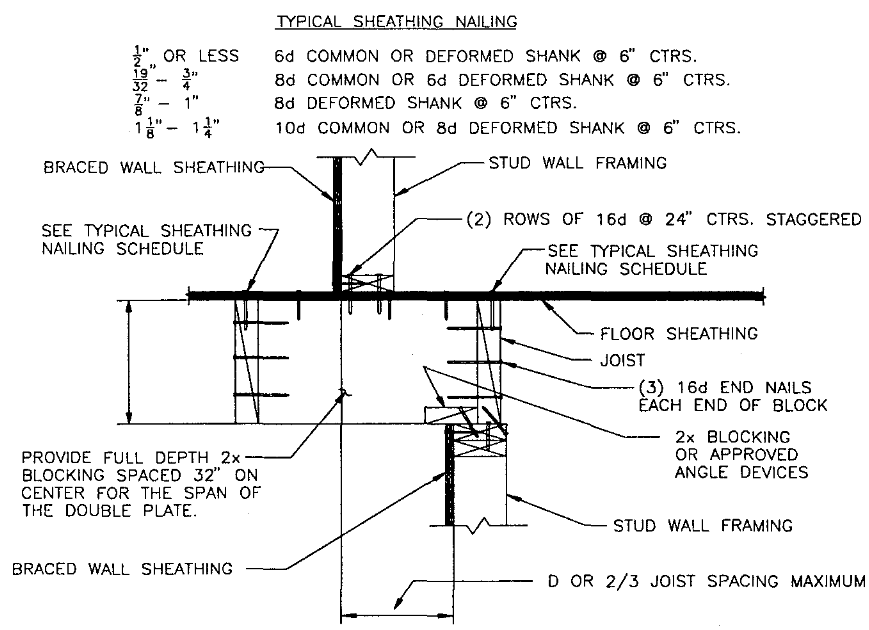


FIGURE C12.5.3-8 Offset at interior braced wall.

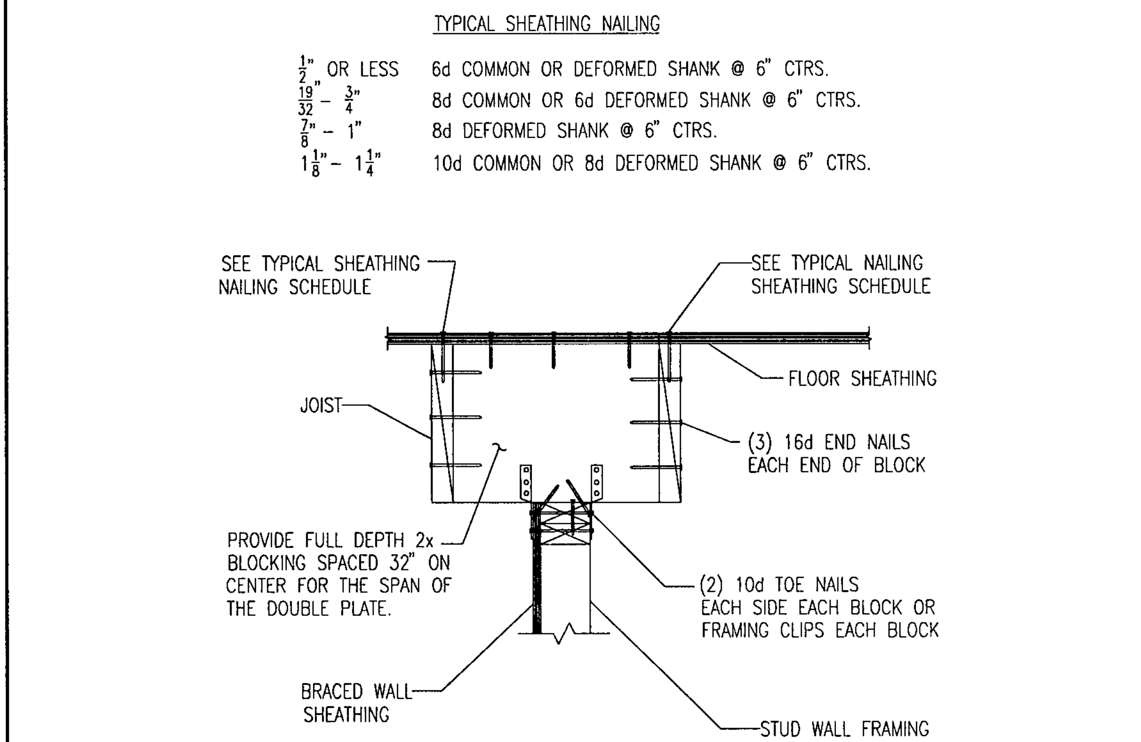


FIGURE C12.5.3-9 Diaphragm connection to braced wall below.

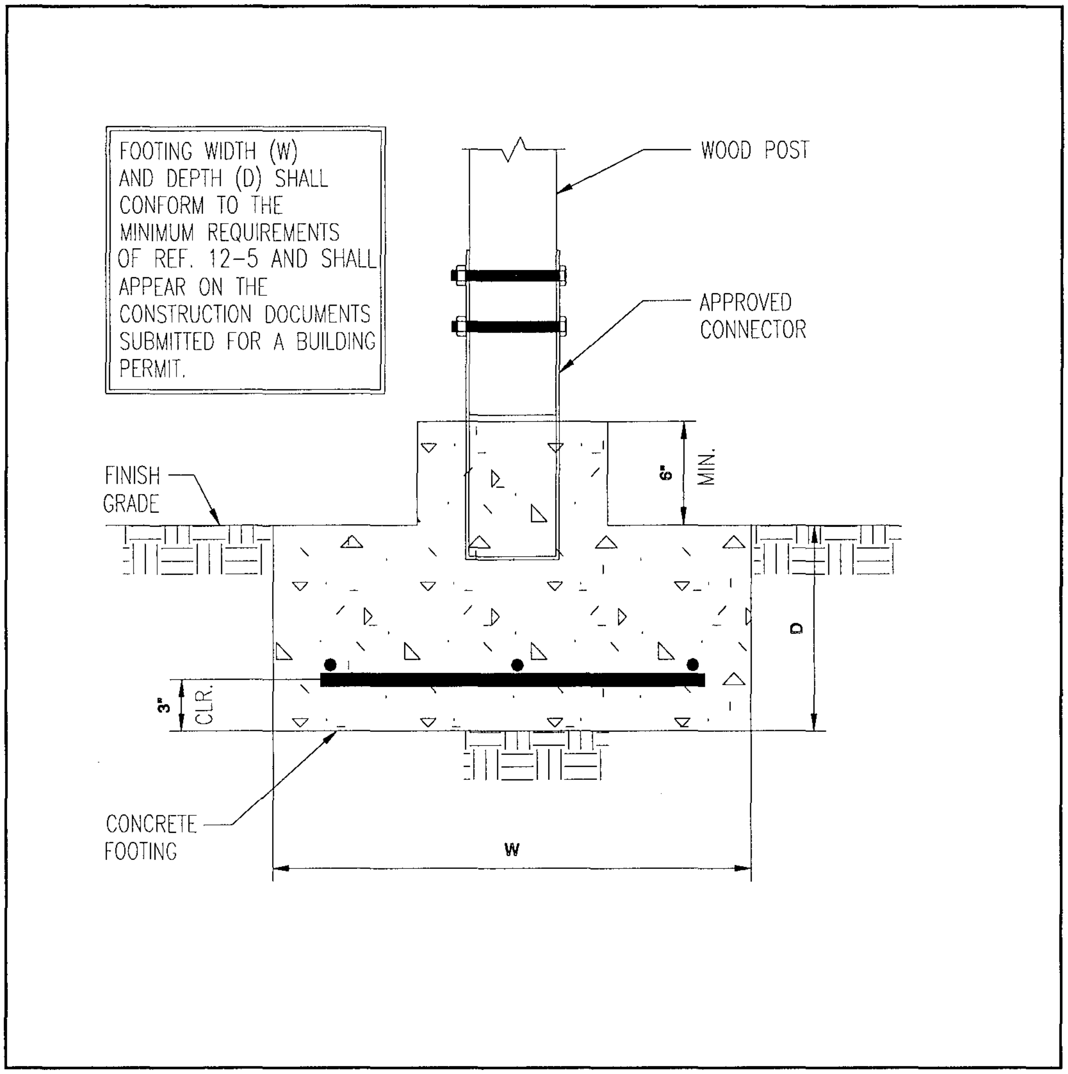


FIGURE C12.5.3-10 Post base detail.

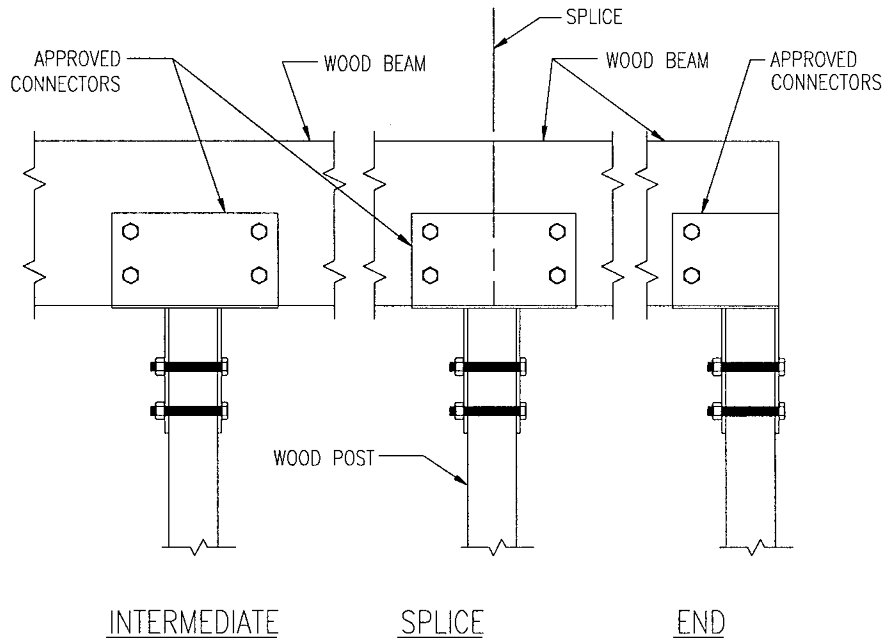


FIGURE C12.5.3-11 Wood beam connection to post.

12.5.3.4 Braced Wall Panel Connections: The exception provided in this section of the *Provisions* is included due to the difficulty in providing a mechanism to transfer the diaphragm loads from a truss roof system to the braced wall panels of the top story. This problem has been considered by the Clackamas County, Oregon Building Codes Division, and an alternate to the CABO Building Code Sec. 402.10 was written in 1993, and revised September 5, 1995. The details shown in Figure C12.5.3.1-1 through C12.5.3.1-4 are provided as suggested methods for providing positive transfer of the lateral forces from the diaphragm through the web sections of the trusses to the top of the braced wall panels below.

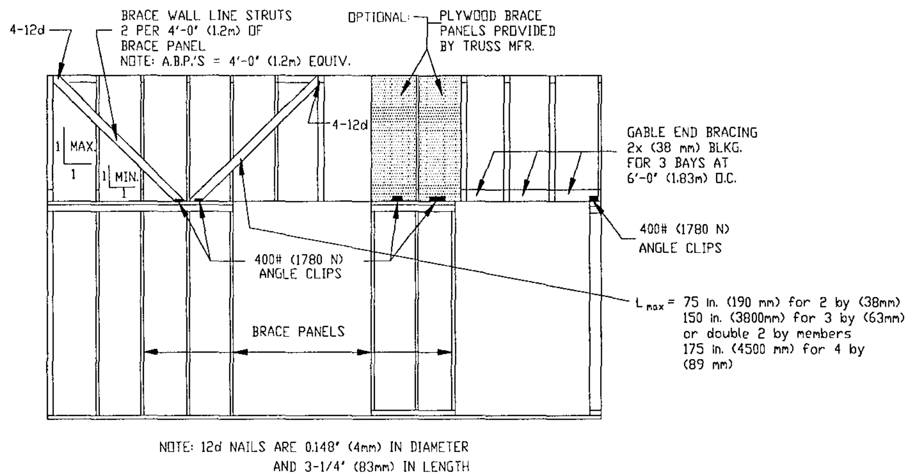


FIGURE C12.5.3.1-1. Suggested methods for transferring roof diaphragms loads to braced wall panels.

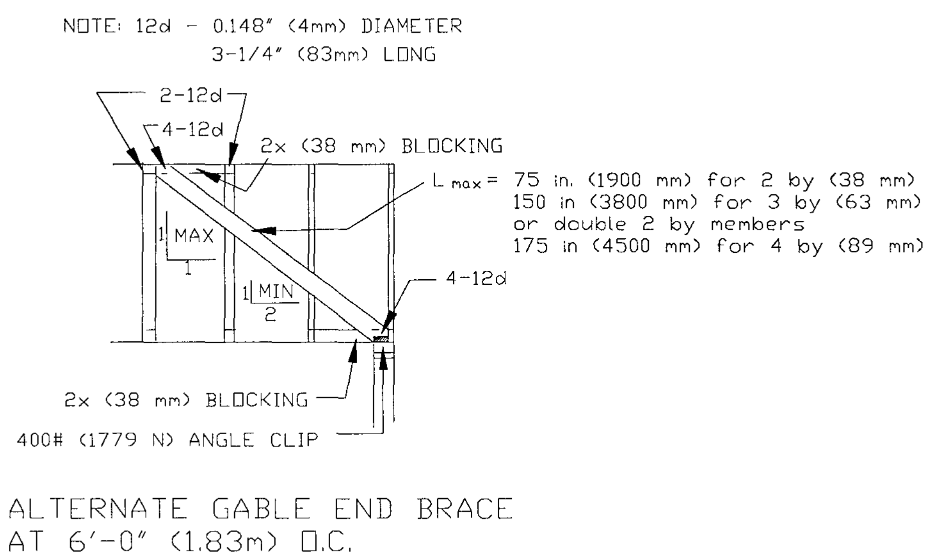


FIGURE C12.5.3.1-2. Alternate gable end brace.

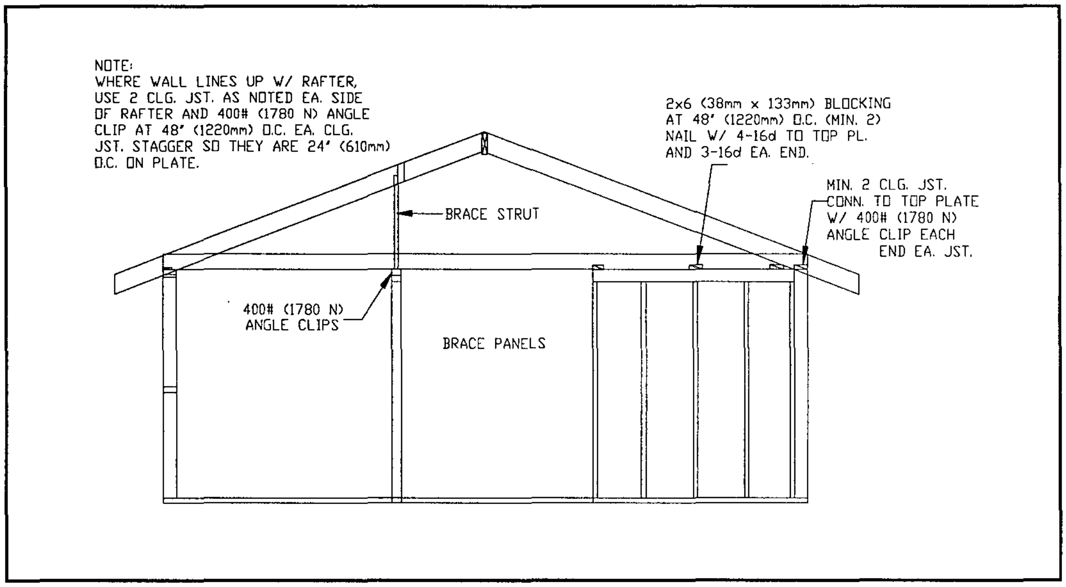


FIGURE C12.5.3.1-3 Wall parallel to truss bracing detail.

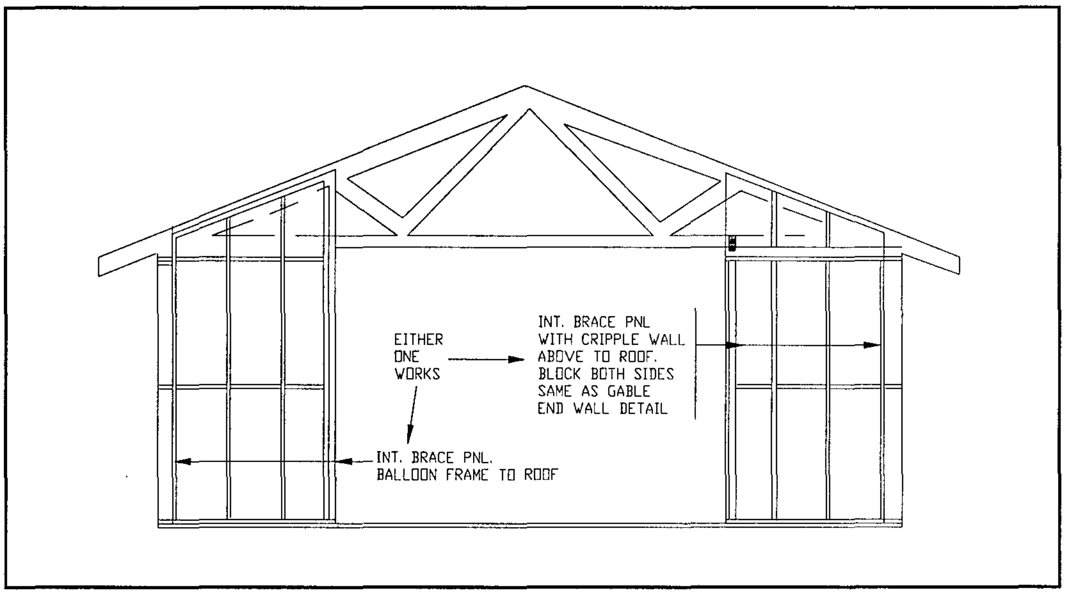


FIGURE C12.5.3.1-4 Wall parallel to truss alternate bracing detail.

12.6 SEISMIC DESIGN CATEGORY A: Wood frame structures assigned to Seismic Design Category A, other than one- and two-family dwellings, must conform with Sec. 12.5 or if engineered need only comply with the reference documents and Sec. 5.2.6.1. Exceptions addressing one- and two-family detached dwellings appear in Sec. 1.2.1.

12.7 SEISMIC DESIGN CATEGORIES B, C, AND D: In the 1997 *Provisions*, Seismic Design Categories B, C, and D, have been combined. At the same time, subsections on material limitations and anchorage requirements have been moved to Sec. 12.3 and 12.4. This was based on the philosophy that detailing requirements should vary based on *R* values, not Seismic Design Categories. Other changes made in the 1997 *Provisions* were editorial (i.e., for clarification or consistency).

Structures assigned to Seismic Design Categories B, C, and D are required to meet the minimum construction requirements of Sec. 12.5 (Sherwood and Stroh, 1989) or must be engineered using standard design methods and principles of mechanics. Conventional light-frame construction requirements were modified in the 1991 *Provisions* to limit the spacing between braced wall lines based on calculated capacities to resist the loads and forces imposed.

Engineered structures assigned to Seismic Design Categories B, C, and D are required to conform to the provisions of Sec. 12.3, Engineered Wood Construction, and Sec. 12.4, Diaphragms and Shear Walls. Included in these sections are general design limitations, limits on wood resisting forces contributed by concrete or masonry, shear wall and diaphragm aspect ratio limitations, and requirements for distribution of shear to vertical resisting elements. See *Commentary* Sec. 12.3 and 12.4.

In the 1997 *Provisions*, Sec. 12.4.1 has been modified to improve the clarity and enforceability of the *Provisions*. The requirements for Seismic Design Categories C and D were moved into the same section as Seismic Design Category B with the triggers for restrictions such as materials limitations associated with Seismic Design Categories C and D being moved to Sec. 12.3 and 12.4.

12.8 SEISMIC DESIGN CATEGORIES E and F: Seismic Design Category F structures require an engineered design. Conventional construction is not considered rigorous enough for structures expected to be functional following a major seismic event. For Seismic Design Category E and F structures, close attention to load path and detailing is required.

Structures assigned to Seismic Design Category E and F require blocked diaphragms. Structural-use panels must be applied directly to the framing members; the use of gypsum wallboard between the structural-use panels and the framing members is prohibited because of the poor performance of nails in gypsum. Restrictions on allowable shear values for structural-use shear panels when used in conjunction with concrete and masonry walls are intended to provide for deformation compatibility of the different materials.

Changes made in the *1997 Provisions* to this section were to provide consistent terminology or were additions taken from the LRFD standard.

REFERENCES

American Institute of Timber Construction. 1994. *Timber Construction Manual.* New York, New York: John Wiley and Sons, Inc.

American Forest & Paper Association. 1991. *ANSI/NFoPA NDS-1991. National Design Specification for Wood Construction.* Washington, D.C.: AF&PA.

American Forest & Paper Association. 1996. *Manual of Wood Construction: Load and Resistance Factor Design (LRFD).* Washington, D.C.: AF&PA.

American Plywood Association. 1991. *Design/Construction Guide--Diaphragms.* Tacoma, Washington: APA.

American Plywood Association. 1994. *Northridge California Earthquake, T94-5.* Tacoma, Washington.: APA.

American Plywood Association. 1996. *Proposed Cyclic Testing Standard for Shear Walls.* Tacoma, Washington: APA.

APA - The Engineered Wood Association. 1996. *Wood structural panel shear walls with gypsum wallboard and window/door openings*, Report 157. Tacoma, Washington: APA.

ASCE Committee on Wood Load and Resistance Factor Design for Engineered Wood Construction. 1988. *A Pre Standard Report American Society of Civil Engineers.* New York, New York: ASCE.

ASCE. 1995. *Standard For Load and Resistance Factor For Engineered Wood Construction.*

AF&PA/ASCE 16-95. American Society of Civil Engineers, New York, New York.

Applied Technology Council. 1981. *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7. Redwood City, California: ATC.

Applied Technology Council. 1995. *Cyclic Testing of Narrow Plywood Shear Walls*, ATC-R-1. Redwood City, California: ATC.

Breyer, Donald E. 1993. *Design of Wood Structures*, Third Edition. New York, New York: McGraw-Hill Book Company.

Canadian Wood Council. 1990. *Wood Design Manual.* Ottawa: Canadian Wood Council.

Canadian Wood Council. 1991. *Wood Reference Handbook.* Ottawa: Canadian Wood Council.

Council of American Building Officials (CABO). 1995. *One- and Two-Family Dwelling Code.* Country Club Hills, Illinois: BOCA.

Department of the Army, Navy and Air Force. 1992. *Seismic Design for Buildings*, TM5-809-10 (Tri-Services Manual). Washington, D.C.: U.S. Government Printing Office.

Dolan, J.D., 1994. "Proposed Test Method for Dynamic Properties of Connections Assembled with Mechanical Fasteners." *ASTM Journal of Testing and Evaluation 22(6):542-547.*

Dolan, J. D., and M. W. White. 1992. "Design Considerations for Using Adhesives in Shear Walls." *ASCE Journal of Structural Engineering* 118(12):3473-3480.

Dolan, J. D. and Johnson, A. C. 1996. "Monotonic Tests of Long Shear Walls with Openings" Virginia Polytechnic Institute and State University, Timber Engineering Report No. TE-1996-001. Blacksburg, Virginia: VPISU.

Dolan, J. D. and Johnson, A. C. 1996. "Cyclic Tests of Long Shear Walls with Openings" Virginia Polytechnic Institute and State University, Timber Engineering Report No. TE-1996-002. Blacksburg, Virginia: VPISU.

Dolan, J. D., and C. P. Heine. 1997a. *Monotonic tests of wood-frame shear walls with various openings and base restraint configurations*, Timber Engineering Report TE-1997-001. Blacksburg: Virginia Polytechnic Institute and State University.

Dolan, J. D., and C. P. Heine. 1997b. *Sequential phased displacement cyclic tests of wood-frame shear walls with various openings and base restraint configurations*, Timber Engineering Report TE-1997-002. Blacksburg: Virginia Polytechnic Institute and State University.

Earthquake Engineering Research Institute. 1996. Northridge earthquake reconnaissance report, Chapter 6, Supplement C to Volume 11, pp 125 et seq., *Earthquake Spectra*.

Faherty, Keith F., and T. G. Williamson. 1989. *Wood Engineering and Construction Handbook*. New York, New York: McGraw-Hill.

U.S. Department of Housing and Urban Development, 1984. *HUD Minimum Property Standards, Vol. I, II, and III.* Washington, D.C.: United States Government Printing Office.

Forest Products Laboratory. 1986. *Wood: Engineering Design Concepts*. University Park: Materials Education Council, The Pennsylvania State University.

Foschi, R. O., and A. Filiatrault. 1990. *Performance Evaluation of 3M Scotch Grip Wood Adhesive 5230 for the Static and Dynamic Design of Timber Shear Walls and Diaphragms*, Vancouver: Department of Civil Engineering Report, University of British Columbia.

Goetz, Karl-Heinz, Dieter Hoor, Karl Moehler, and Julius Natterer. 1989. *Timber Design and Construction Source Book: A Comprehensive Guide to Methods and Practice*. New York, New York: McGraw-Hill.

Hoyle and Woeste. 1989. *Wood Technology and Design of Structures*. Iowa State University Press.

International Conference of Building Officials (ICBO). 1994. *Uniform Building Code*. Whittier, California: ICBO.

Keenan, F. J. 1986. *Limit States Design of Wood Structures*. Morrison Hershfield Limited.

NAHB Research Center. 1998. *The performance of shear walls with narrow wall segments, reduced based restraint, and alternative framing methods.* Upper Marlboro, Maryland.

Rose, J. D., 1996. Research Report 158. "Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading." Tacoma, Washington: APA.

Sherwood and Stroh. 1989. "Wood-Frame House Construction" in *Agricultural Handbook 73*. Washington, D.C.: U.S. Government Printing Office.

Somayaji, Shan. 1992. *Structural Wood Design*. St. Paul, Minnesota: West Publishing Co.

Stalnaker, Judith J., and E. C. Harris. 1996. *Structural Design in Wood,* Second Edition. New York, New York: McGraw-Hill.

Sugiyama, H. 1981. "The evaluation of shear strength of plywood-sheathed walls with openings," *Wood Industry*, 36-7, July.

Sugiyama, H., and T. Matsumoto. 1994. "Empirical equations for the estimation of racking strength of plywood-sheathed shear wall with openings," in *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, September.

U.S. Department of Agriculture, National Oceanic and Atmospheric Administration. 1971. *San Fernando, California, Earthquake of February 9, 1971*. Washington, D.C.: NOAA.

White, M. W. and J. D. Dolan, 1996. "Seismic Response of Timber Shear Walls. Part I: Aspect Ratios." Paper submitted for publication in *ASCE Journal of Structural Engineering*.

Yasumura, M., and H. Sugiyama. 1984. "Shear properties of plywood-sheathed wall panels with openings," in *Transaction of the Architectural Institute of Japan*, No. 338, April.

Additional references are listed in John Peterson. 1983. "Bibliography on Lumber and Wood Panel Diaphragms" , *Journal of Structural Engineering*, Vol. 109 No. 12. American Society of Civil Engineers, New York, New York.

Chapter 13 Commentary

SEISMICALLY ISOLATED STRUCTURES DESIGN REQUIREMENTS

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By decoupling the structure from the ground motion, the level of response in the structure can be significantly reduced from the level that would otherwise occur in a conventional fixed-base building. Conversely, seismic isolation permits designing with a reduced level of earthquake load to achieve the same degree of seismic protection and reliability as a conventional fixed-base building.

The potential advantages of seismic isolation and the recent advancements in isolation-system products already have led to the design and construction of over 100 seismically isolated buildings and bridges in the United States. A significant amount of research, development, and application activity has occurred over the past 20 years. The following references provide a summary of some of the work that has been performed: Applied Technology Council (1986, 1993), ASCE Structures Congress (1989, 1991, 1993 and 1995), EERI Spectra (1990), Skinner, et al. (1993), U.S. Conference on Earthquake Engineering (1990 and 1994), and World Conference on Earthquake Engineering (1988, 1992 and 1996).

In the mid-1980s, the initial applications identified a need to supplement existing codes with design requirements developed specifically for seismically isolated buildings. Code development work occurred throughout the late 1980s. The status of U.S. seismic isolation design requirements as of October 1996 is as follows:

1. In late 1989, the Structural Engineers Association of California (SEAOC) State Seismology Committee adopted an "Appendix to Chapter 2" of the SEAOC Blue Book entitled, "General Requirements for the Design and Construction of Seismic-Isolated Structures." These requirements were submitted to the International Conference of Building Officials (ICBO) and were adopted by ICBO as an appendix of the 1991 *Uniform Building Code* (*UBC*). The isolation appendix of the *UBC* has been updated on an annual basis since that time and the most current version of these regulations may be found in the 1997 *UBC*.

2. In the late 1980s, the *building* Safety Board (BSB) of California, Office of the State Architect, adopted *An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation* based on recommendations of SEAOC. These methods were used for regulation of California hospitals until the BSB replaced them with the 1991 *UBC* appendix (with slight modification). The current version of these regulations may be found in 1995 *California Building Code.*

3. In 1991 the Federal Emergency Management Agency (FEMA) initiated a 6-year program to develop a set of nationally applicable guidelines for seismic rehabilitation of existing buildings. These guidelines (known as the *NEHRP Guidelines for the Seismic Rehabilitation*

of Buildings) are now available as FEMA 273. The design and analysis methods of the *NEHRP Guidelines* parallel closely methods required by the *NEHRP Recommended Provisions* for new buildings, except that more liberal design is permitted for the superstructure of a rehabilitated building.

During development of the 1994 *Provisions*, it was decided to use the latest version (1993 approved changes) of the SEAOC/*UBC* provisions as a basis for the development of the requirements included in the *Provisions*. The only significant changes involved an appropriate conversion to strength design and making the requirements applicable on a national basis. For the 1997 *Provisions*, it was decided to incorporate the latest version of the SEAOC/*UBC* provisions (1997 *UBC*). Since the 1997 *UBC* is now based on strength design, the 1997 *UBC* and the 1997 *Provisions* are almost identical, except for seismic criteria. The seismic criteria of the *Provisions* are based on the new national earthquake maps (developed by the Seismic Design Procedures Group) which can be substantially different from the seismic criteria of the 1997 *UBC*.

A general concern has long existed regarding the applicability of different types of isolation systems. Rather than addressing a specific method of base isolation, the *Provisions* provides general design requirements applicable to a wide range of possible seismic isolation systems. Although remaining general, the design requirements rely on mandatory testing of isolation-system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will: (1) remain stable for required design displacements, (2) provide increasing resistance with increasing displacement, (3) not degrade under repeated cyclic load, and (4) have quantifiable engineering parameters (e.g., force-deflection characteristics and damping).

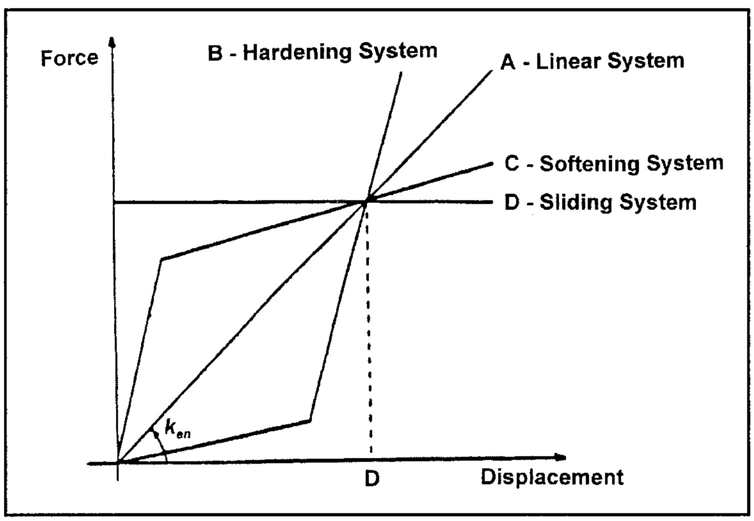


FIGURE C13 Idealized force-deflection relationships for isolation systems (stiffness effects of sacrificial wind-restraint systems not shown for clarity).

Conceptually, there are four basic types of isolation system force-deflection relationships. These idealized relationships are shown in Figure C13 with each idealized curve having the same design displacement, D_D, for the design earthquake. A linear isolation sys tem is represented by Curve A and has the same isolated period for all earthquake load levels. In addition, the force generated in the superstructure is directly proportional to the displacement across the isolation system.

A hardening isolation system is represented by Curve B. This system is soft initially (long effective period) and then stiffens (effective period shortens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a hardening system, the superstructure is subjected to higher forces and the isolation system to lower displacements than a comparable linear system.

A softening isolation system is represented by Curve C. This system is stiff initially (short effective period) and softens (effective period lengthens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a softening system, the superstructure is subjected to lower forces and the isolation system to higher displacements than a comparable linear system.

A sliding isolation system is represented by Curve D. This system is governed by the friction force of the isolation system. Like the softening system, the effective period lengthens as the earthquake load level increases and loads on the superstructure remain constant.

The total system displacement for extreme displacement of the sliding isolation system, after repeated earthquake cycles, is highly dependent on the vibratory characteristics of the ground motion and may exceed the design displacement, D_D. Consequently, minimum design requirements do not adequately define peak seismic displacement for seismic isolation systems governed solely by friction forces.

13.1 GENERAL: The design requirements permit the use of one of three different analysis procedures for determining the design-basis seismic loads. The first procedure uses a simple-lateral-force formula (similar to the lateral-force coefficient now used in conventional building design) to prescribe peak lateral displacement and design force as a function of spectral acceleration and isolated-building period and damping. The second and third methods, which are required for geometrically complex or especially flexible buildings, rely on dynamic analysis procedures (either response spectrum or time history) to determine peak response of the isolated building.

The three procedures are based on the same level of seismic input and require a similar level of performance from the building. There are benefits in performing a more complex analysis in that slightly lower design forces and displacements are permitted as the level of analysis becomes more sophisticated. The design requirements for the structural system are based on the design earthquake, a severe level of earthquake ground motion defined as two-thirds of the maximum considered earthquake. The isolation system, including all connections, supporting structural elements and the "gap," is required to be designed (and tested) for 100 percent of maximum considered earthquake demand. Structural elements above the isolation system are not required to be designed for the full effects of the design earthquake , but may be designed for slightly reduced loads (i.e., loads reduced by a factor of up to 2.0) if the structural system has sufficient ductility, etc., to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

Ideally, lateral displacement of an isolated structure will result, predominantly due to the deformations of the isolation system, rather than in distortion of the structure above. Accordingly, the lateral-load-resisting system of the structure above the isolation system should be designed to have sufficient stiffness and strength to avoid large, inelastic displacements. For this reason, the *Provisions* contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control for the design-basis earthquake is not an explicit objective of the *Provisions*, an isolated structure designed to limit inelastic response of the structural system also will reduce the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in conformance with the *Provisions* should be able to:

1. Resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural *components*, or building contents and
2. Resist major levels of earthquake ground motion without failure of the isolation system, without significant damage to structural elements, without extensive damage to nonstructural *components*, and without major disruption to facility function.

The above performance objectives for isolated structures considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table C13.1 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with the *Provisions*. Loss of function is not included in Table C13.1. For certain (fixed-base) facilities, loss of function would not be expected to occur until there is significant structural damage causing closure or restricted access to the building. In other cases, the facility could have only limited or no structural damage but would not be functional as a result of damage to vital nonstructural *components* and contents. Isolation would be expected to mitigate structural and nonstructural damage and protect the facility against loss of function.

The requirements of Chapter 13 provide isolator design displacements, structure-design-shear forces, and other specific requirements for seismically isolated structures. All other design requirements including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal-shear distribution are covered by the applicable sections of the *Provisions* for conventional fixed-base structures.

TABLE C13.1 Protection Provided by *NEHRP Recommended Provisions* for Minor, Moderate and Major Levels of Earthquake Ground Motion

Risk Category	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life safety[a]	F/I	F/I	F/I
Structural damage[b]	F/I	F/I	I
Nonstructural damage[c] (contents damage)	F/I	I	I

[a] Loss of life or serious injury is not expected for fixed-*base* (F) or isolated (I) *buildings*.

[b] Significant structural damage is not expected for fixed-*base* (F) or isolated (I) *buildings*.

[c] Significant nonstructural (contents) damage is not expected for fixed-*base* (F) or isolated (I) *buildings*.

13.2 CRITERIA SELECTION: This section delineates the requirements for the use of the equivalent-lateral-force and dynamic methods of analysis and the conditions for developing a site-specific response spectrum. The limitations on the simplified lateral-force design procedure are quite severe at this time. Limitations cover the site location with respect to active faults; soil conditions of the site, the height, regularity and stiffness characteristics of the building; and the characteristics of the isolation system. In fact, the current limitations will necessitate a dynamic analysis for most isolated structures. Additionally, time-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement;
2. Isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the *Provisions* for "essentially-elastic" design; and
3. Isolated structures located on Class F site. (i.e., very soft soil).

The restrictions placed on the use of equivalent-lateral-force design procedures effectively require dynamic analysis for virtually all isolated structures. However, lower-bound limits on isolation system design displacements and structural-design forces are specified by the *Provisions* in Sec. 13.4 as a percentage of the values prescribed by the equivalent-lateral-force design formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters ensure consistency in the design of isolated structures and serve as a "safety net" against gross under-design. Table C13.2 provides a summary of the lower-bound limits on dynamic analysis specified by the *Provisions.*

TABLE C13.2 Lower-Bound Limits on Dynamic Analysis Specified as a Percentage of Static-Analysis Design Requirements

Design Parameter	Static Analysis	Dynamic Analysis	
		Response Spectrum	Time History
Design Displacement - D_D	$D_D = (g/4\pi^2)(S_{D1}T_D/B_D)$	–	–
Total Design Displacement - D_T	$D_T \geq 1.1D$	$\geq 0.9D_T$	$\geq 0.9D_T$
Maximum Displacement - D_M	$D_M = (g/4\pi^2)(S_{M1}T_M/B_M)$	–	–
Total Maximum Displacement - D_{TM}	$D_{TM} \geq 1.1D_M$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design Shear - V_b (at or below the *Isolation System*)	$V_b = k_{Dmax}D_D$	$\geq 0.9V_b$	$\geq 0.9V_b$
Design Shear - V_s ("Regular" Superstructure)	$V_s = k_{Dmax}D_D/R_I$	$\geq 0.8V_s$	$\geq 0.6V_s$
Design Shear - V_s ("Irregular" Superstructure)	$V_s = k_{Dmax}D_DR_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Drift (calculated using R_I for C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

Site-specific design spectra must be developed for both the design earthquake and the maximum considered earthquake if the *structure* is located at a site with S_1 greater than 0.60g or on a Class

F site. Lower limits are placed on these site-specific spectra and they must not be less than 80 percent of those given in Sec. 13.4.4.

13.3 EQUIVALENT LATERAL FORCE PROCEDURE: The lateral displacement given by Equation 13.3.3.1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_D, and equivalent viscous damping, β_D, and the lateral displacement given by Equation 13.3.3.3 approximates peak maximum considered earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_M, and equivalent viscous damping, β_{DM}.

13.3.3 Minimum Lateral Displacements: Equation 13.3.3.1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, S_{D1}, is the same as that required for design of a conventional fixed-base structure of period, T_D. A damping term, B_D, is used to decrease (or increase) the computed displacement when the equivalent damping coefficient of the isolation system is greater (or smaller) than 5 percent of critical damping. Values of coefficient, B_D (or B_M for the maximum considered earthquake), are given in Table 13.3.3.1. for different values of isolation system damping, β_D (or β_M).

A comparison of values obtained from Equation 13.3.3.1 and those obtained from nonlinear time-history analyses are given in references by Kircher et al. (1988), Lashkari and Kircher (1993) and Constantinou et al. (1993).

Consideration should be given to possible differences in the properties of the isolation system used for design and the properties of isolation system actually installed in the building. Similarly, consideration should be given to possible changes in isolation system properties due to different design conditions or load combinations. If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (i.e., rate, amplitude or time dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection (k_{Dmin}) and the design forces should be based on deformational characteristics of the isolation system that give the largest possible force (k_{Dmax}). If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (i.e., rate, amplitude or time dependent), the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

The configuration of the isolation system for a seismically isolated building or structure should be selected in such a way as to minimize any eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements will be reduced. As for conventional structures, allowance for accidental eccentricity in both horizontal directions must be considered. Figure C13.3.3 defines the terminology used in the *Provisions*. Equation 13.3.3.5-1 (or Equation 13.3.3.5-2 for the maximum considered earthquake) provides a simplified formulae for estimating the response due to torsion in lieu of a more refined analysis. The additional component of displacement due to torsion increases the design displacement at the corner of the structure by about 15 percent (for a perfectly square building in plan) to about 30 percent (for a very long, rectangular building) if the eccentricity is 5 percent of the maximum plan dimension. Such additional displacement, due to torsion, is appropriate for buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building or certain sliding systems that minimize the effects of mass eccentricity will have reduced displacements due to torsion. The *Provisions* permits values of D_T as small as $1.1D_D$, with proper justification.

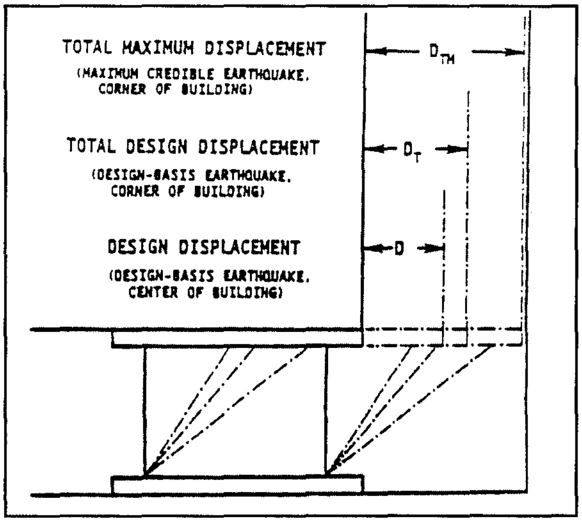


FIGURE C13.3.3 Displacement terminology.

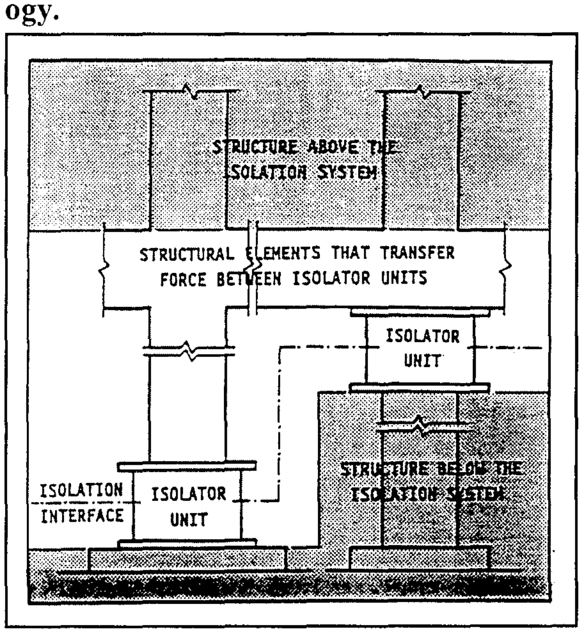


FIGURE C13.3.4 Isolation system terminology.

13.3.4 Minimum-Lateral Forces: Figure C13.3.4 defines the terminology below and above the *isolation system*. Equation 13.3.4.1 gives peak seismic shear on all structural *components* at or below the seismic interface without reduction for ductile response. Equation 13.3.4.2 specifies the peak seismic shear for design of structural systems above the seismic interface. For *structures* that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

The basis for the reduction factor is that the design of the structural system is based on strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of 10 specific buildings indicated that this factor varied between 2 and 5 (Applied Technology Council, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for the design earthquake .

In Sec. 13.3.4.3, the limitations given on V_S ensure that there is at least a factor of 1.5 between the nominal yield level of the superstructure and (1) the yield level of the isolation system, (2) the ultimate capacity of a sacrificial-wind-restraint system which is intended to fail and release the superstructure during significant lateral load, or (3) the break-away friction level of a sliding system.

These limitations are essential to ensure that the superstructure will not yield prematurely before the isolation system has been activated and significantly displaced.

The design shear force, V_S, specified by the requirements of this section ensures that the structural system of an isolated building will be subjected to significantly less inelastic demands than a conventionally designed structure. Further reduction in V_S, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

If the level of performance of the isolated *structure* is desired to be greater than that implicit in these requirements, then the denominator of Equation 13.3.4.2 may be reduced. Decreasing the denominator of Eq. 13.3.4.2 will lessen or eliminate inelastic response of the superstructure for the design-basis event.

13.3.5 Vertical Distribution of Force: Equation 13.3.5 describes the vertical distribution of lateral force based on an assumed triangular distribution of seismic acceleration over the height of the structure above the isolation interface. References by Button (1993) and Constantinou et al. (1993) provide a good summary of recent work which demonstrates that this vertical distribution of force will always provide a conservative estimate of the distributions obtained from more-detailed-nonlinear analysis studies.

13.3.6 Drift Limits: The maximum interstory drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table C13.3.6. For comparison, the drift limits prescribed by the *Provisions* for fixed-base structures also are summarized in Table C13.3.6.

TABLE C13.3.6 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Seismic Use Group	Fixed-Base	Isolated
Buildings (other than masonry) four stories or less in height with component drift design	I	$0.025h_{sx}/(C_d/R)$	$0.015h_{sx}$
	II	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
Other (non-masonry) buildings	I	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	II	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.010h_{sx}/(C_d/R)$	$0.015h_{sx}$

Drift limits in Table C13.3.6 are divided by C_d/R for fixed-*base structures* since *displacements* calculated for lateral loads reduced by R. are factored by C_d before checking drift. The C_d term is used throughout the *Provisions* for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for "reduced" forces. Generally, C_d is $\frac{1}{2}$ to $\frac{4}{5}$ the value of R. For isolated structures, the R_I factor is used both to reduce lateral loads and to increase displacements (calculated for reduced lateral loads) before checking drift. Equivalency would be obtained if the drift limits for both fixed-base and isolated structures were based on their respective R factors. It may be note that the drift limits for isolated structures are generally more conservative than those of conventional fixed-base structures, even when fixed-base structures are designed as Seismic Use Group III buildings.

13.4 DYNAMIC LATERAL RESPONSE PROCEDURE: This section specifies the requirements and limits of a dynamic analysis. The design displacement and force limits on a response-spectrum and time-history analysis are given in Table C13.2.

A more-detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide analysis procedures which are compatible with the minimum requirements of Sec. 13.3. Reasons for performing a more-refined study include:

1. The importance of the building.
2. The need to analyze possible structure/isolation-system interaction when the fixed-base period of the building is greater than one third of the isolated period.
3. The need to explicitly model the deformational characteristics of the lateral-force-resisting system when the structure above the isolation system is irregular.
4. The desirability of using site-specific ground-motion data, especially for soft soil types (Site Class F) or for *structures* located on sites with S_1 greater than 0.60g.
5. The desirability of explicitly modeling the deformational characteristics of the base-isolation system. This is especially important for systems that have damping characteristics that are amplitude, rather than velocity, dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Additionally, time-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement.
2. Isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the SEAOC/UBC provisions for "essentially-elastic" design.
3. Isolated structures located on Class F sites (i.e., very soft soil).

When time-history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are to be based on the maximum of the results of not less than three separate analyses, each using a different pair of horizontal time histories. Each pair of horizontal time histories is to:

1. Be of a duration consistent with the design earthquake or the maximum considered earthquake,
2. Incorporate near-field phenomena, as appropriate, and

3. Have response spectra whose square-root-sum-of-the-squares combination of the two horizontal *components* equals or exceeds 1.3 times the "target" spectrum at each spectral ordinate.

The average value of seven time histories is a standard required by the nuclear industry and is considered appropriate for nonlinear time-history analysis of seismically isolated structures.

13.5 LATERAL LOAD ON ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS SUPPORTED BY BUILDINGS: To accommodate the differential movement between the isolated building and the ground, provision for flexible utility connections should be made. In addition, rigid structures crossing the interface, (i.e., stairs, elevator shafts and walls, should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

13.6 DETAILED SYSTEM REQUIREMENTS: Environmental conditions that may adversely effect isolation system performance should be thoroughly investigated. Significant research has been conducted on the effects of temperature, aging, etc., on isolation systems since the 1970s in Europe, New Zealand, and the United States.

13.6.2.2 Wind Forces: Lateral displacement over the depth of the isolator zone resulting from wind loads should be limited to a value similar to that required for other story heights.

13.6.2.3 Fire Resistance: In the event of a fire, the isolation system should be capable of supporting the weight of the building, as required for other vertical-load-supporting elements of the structure, but may have diminished functionality for lateral (earthquake) load.

13.6.2.4 Lateral Restoring Force: The isolation system should be configured with a lateral-restoring force sufficient to avoid significant residual displacement as a result of an earthquake, such that the isolated structure will not have a stability problem and be in a condition to survive aftershocks and future earthquakes.

13.6.2.5 Displacement Restraint: The use of a displacement restraint is not encouraged by the *Provisions*. Should a displacement restraint system be implemented, explicit analysis of the isolated structure for maximum considered earthquake is required to account for the effects of engaging the displacement restraint.

13.6.2.6 Vertical Load Stability: The vertical loads to be used in checking the stability of any given isolator should be calculated using bounding values of dead load and live load and the peak earthquake demand of the maximum considered earthquake. Since earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner which produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at peak maximum considered earthquake displacement of the isolation system.

13.6.2.7 Overturning: The intent of this requirement is to prevent global, structural overturning and overstress of elements due to local uplift. Uplift in a braced frame or shear wall is acceptable, provided the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. If the tension capacity of an isolation system is to be utilized on resisting uplift forces, then component tests should be performed to demonstrate the adequacy of the system on resisting-tension forces at the design displacement.

13.6.2.8 Inspection and Replacement: Although most isolation systems will not need to be replaced after an earthquake, it is good practice to provide for inspection and replacement. After an earthquake, the building should be inspected and any damaged elements should be replaced or repaired. It is advised that periodic inspections be made of the isolation system.

13.6.2.9 Quality Control: A test and inspection program is necessary for both fabrication and installation of the isolation system. Because base isolation is a developing technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials such as elastomeric bearings (ASTM D4014). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should be developed for each project. The requirements will vary with the type of isolation system used.

13.6.3 Structural System:

13.6.3.2 Building Separations: A minimum separation between the isolated structure and a rigid obstruction is required to allow free movement in all lateral directions of the superstructure during an earthquake. Provision should be made for lateral motion greater than the design displacement, since the exact upper limit of displacement cannot be precisely determined.

13.8 DESIGN AND CONSTRUCTION REVIEW: Design review of the design and analysis of the isolation system and design review of the isolator testing program is mandated by the *Provisions* for two key reasons:

1. The consequences of isolator failure could be catastrophic.
2. Isolator design and fabrication technology is evolving rapidly and may be based on technologies unfamiliar to many design professionals.

The *Provisions* requires review to be performed by a team of registered design professionals that are independent of the design team and other project contractors. The review team should include individuals with special expertise in one or more aspects of the design, analysis and implementation of seismic isolation systems.

The review team should be formed prior to the development of design criteria (including site-specific ground shaking criteria) and isolation system design options. Further, the review team should have full access to all pertinent information and the cooperation of the design team and regulatory agencies involved with the project.

13.9 REQUIRED TESTS OF THE ISOLATION SYSTEM: The design displacements and forces developed from the *Provisions* are predicated on the basis that the deformational characteristics of the base isolation system have been previously defined by a comprehensive set of tests. If a comprehensive amount of test data are not available on a system, then major design alterations in the building may be necessary after the tests are complete. This would result from variations in the isolation-system properties assumed for design and those obtained by test.

Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data is not available on an isolation system.

Typical force-deflection or hysteresis loops are shown in Figure C13.9; also included are the definitions of values used in Sec. 13.9.3.

The required sequence of tests will experimentally verify:

1. The assumed stiffness and capacity of the wind-restraining mechanism;
2. The variation in the isolator's deformational characteristics with amplitude and with vertical load, if it is a vertical load-carrying member;
3. The variation in the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and
4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.

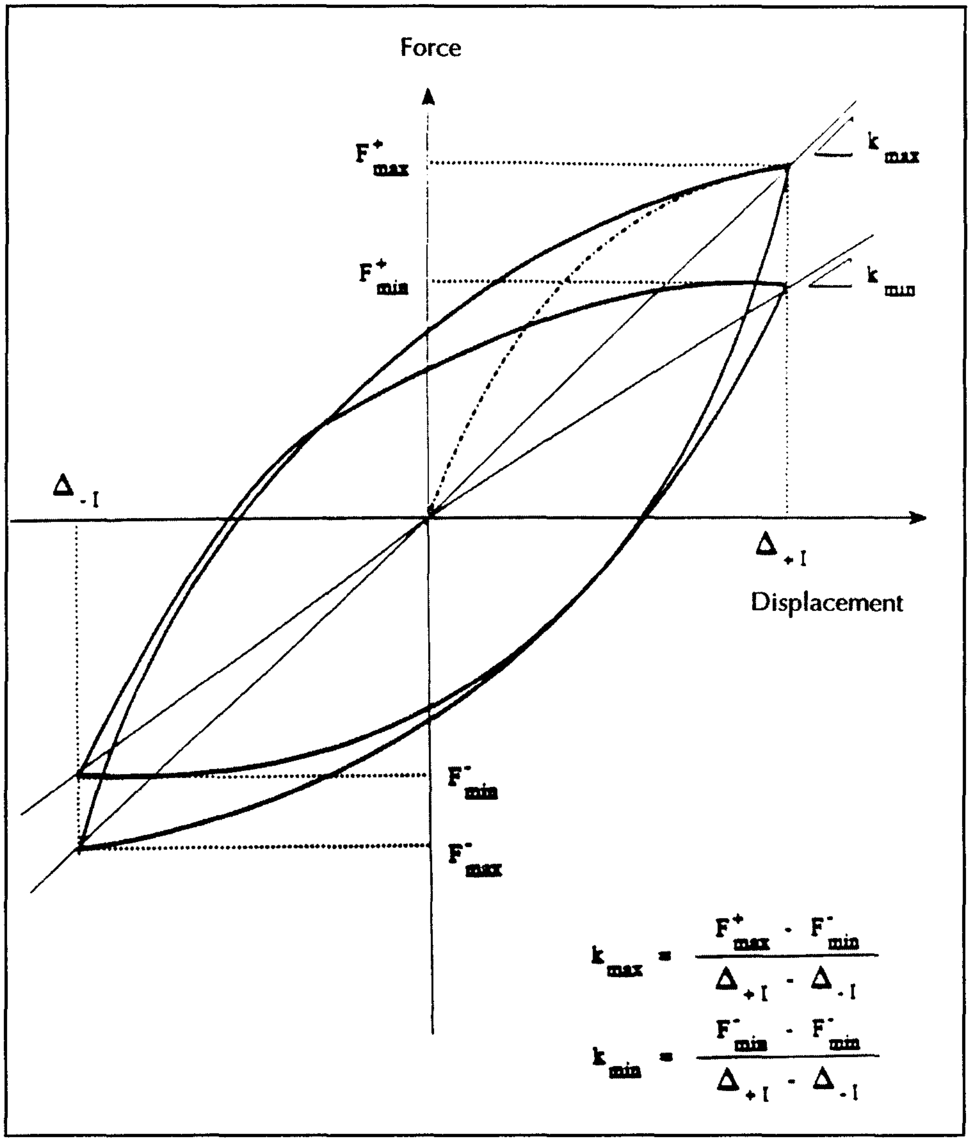


FIGURE 13.9 The effect of stiffness on an isolation bearing.

Force-deflection tests are not required if similarly sized *components* have been previously tested using the specified sequence of tests.

Variations in effective stiffness greater than ±15 percent over 3 cycles of loading at a given amplitude, or ±20 percent over the larger number of cycles at the design displacement, would be cause for rejection. The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral, load are necessary to determine possible variations in the system properties with variations in overturning force. The appropriate dead loads and overturning forces for the tests are defined as the average loads on a given type and size of isolator for determining design properties and are the absolute maximum and minimum loads for the stability tests.

13.9.5 Design Properties of the Isolated System:

13.9.5.1 Maximum and Minimum Effective Stiffness: The effective stiffness is determined from the hysteresis loops shown in Figure C13.9). Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (±15 percent) for more than 3 cycles at a given amplitude.

The intent of these requirements is to ensure that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the lowest damping and effective-stiffness values. For determining design forces, this means using the lowest damping value and the greatest stiffness value.

13.9.5.2 Effective Damping: The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent, energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous-damping value. Since it is difficult to relate velocity and amplitude-dependent phenomena, it is recommended that when the equivalent-viscous damping assumed for the design of amplitude-dependent, energy-dissipating mechanisms (e.g., pure-sliding systems) is greater than 30 percent, then the design-basis force and displacement should be determined by the time-history-analysis method, as specified in Sec. C13.2.

REFERENCES

Applied Technology Council. 1982. *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, ATC Report 10. Redwood City, California: ATC

Applied Technology Council. 1986. *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, ATC Report 17. Redwood City, California: ATC.

Applied Technology Council. 1993. *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, ATC 17-1. Redwood City, California: ATC.

American Society of Civil Engineers. 1989, 1991, 1993, and 1995. *Seismic Engineering: Research and Practice*. New York City: ASCE.

Constantinou, M. C., C. W. Winters, and D. Theodossiou. 1993. "Evaluation of SEAOC and UBC analysis procedures, Part 2: Flexible superstructure," in *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, ATC Report 17-1. Redwood City, California: ATC.

Earthquake Engineering Research Institute. 1990. "Seismic isolation: from idea to reality," *Earthquake Spectra Journal* 6:2.

Kircher, C. A., B. Lashkari, R. L. Mayes, and T. E. Kelly. 1988. "Evaluation of nonlinear response in seismically isolated buildings," in *Proceedings of a Symposium on Seismic, Shock and Vibration Isolation*, ASMA PVP Conference.

Lashkari, B., and C. A. Kircher. 1993. "Evaluation of SEAOC & UBC analysis procedures, part 1: stiff superstructure," in *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, ATC Report 17-1. Redwood City, California: ATC.

Lashkari, B., and C. A. Kircher. 1990. *Proceedings of Fourth U.S. National Conference on Earthquake Engineering*. Berkeley, California: Earthquake Engineering Research Institute.

Skinner, R. I., W. H. Robinson, and G. H. McVerry. 1993. *An Introduction to Seismic Isolation*. Sussex, England: Wiley and Sons.

Appendix to Chapter 13 *Commentary*

STRUCTURES WITH DAMPING SYSTEMS

Appendix A13 is an entirely new addition to the 2000 *Provisions* that does not include a detailed commentary at this time. A detailed commentary will be developed during the next update cycle when it is expected that the appendix will be incorporated into the main body of the *Provisions.*

The balance of this section provides background on the underlying philosophy used by TS-12 to develop the appendix, the definition of the damping system, the concept of effective damping, and the calculation of earthquake response using linear analysis methods.

The basic approach taken by TS-12 in developing the appendix for *structures* with *damping systems* is based on the following concepts:

1. Appendix is applicable to all types of *damping systems*, including both *displacement-dependent damping devices* of hysteretic or friction systems and *velocity-dependent damping devices* of viscous or visco elastic systems.
2. Appendix provides minimum design criteria with performance objectives comparable to those of a *structure* with a conventional *seismic-force-resisting system* (but also permits design criteria that will achieve higher performance levels).
3. Appendix requires *structures* with a *damping system* to have a *seismic-force-resisting system* that provides a complete load path. The *seismic-force-resisting system* must comply with the requirements of the *Provisions*, except that the *damping system* may be used to meet drift limits.
4. Appendix requires design of *damping devices* and prototype testing of damper units for displacements, velocities and forces corresponding to those of the *maximum earthquake* (same approach as that used for *structures* with an *isolation system*).
5. Appendix provides "simple" linear static or response spectrum analysis methods for design of most *structures* that meet certain configuration and other limiting criteria (e.g., at least two *damping devices* at each story configured to resist torsion). *Appendix requires additional nonlinear time history analysis to confirm peak response of structures not meeting the criteria for linear analysis (and for structures close to faults).*

Damping System: The appendix defines the *damping system* as:

> The collection of structural elements that includes all individual *damping devices*, all structural elements or bracing required to transfer forces from damping devices to the *base* of the *structure* and all structural elements required to transfer forces from *damping devices* to the *seismic-force-resisting system.*

The *damping system* is defined separately from the *seismic-force-resisting system*, although the two systems may have common elements. As illustrated in Figure CA13-1, the *damping system* may be external or internal to the *structure* and may have no shared elements, some shared

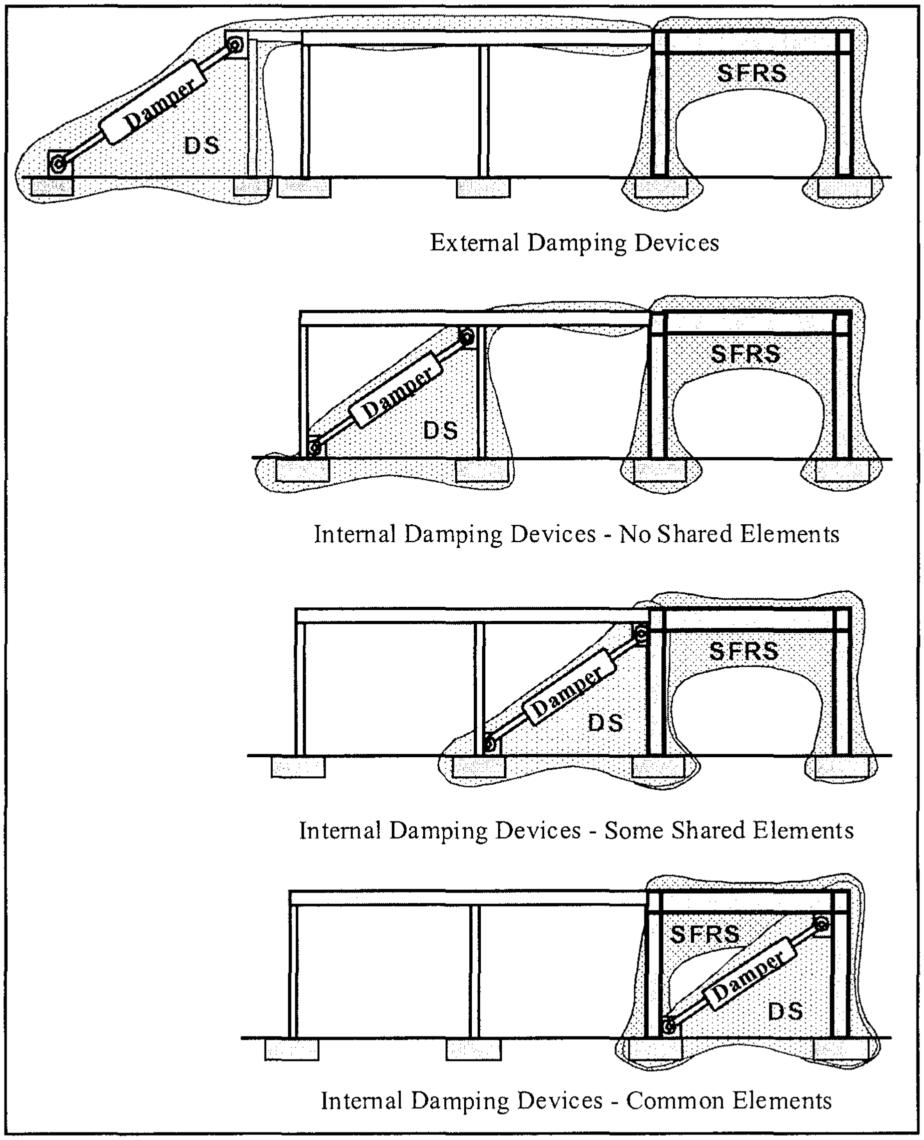


FIGURE C13A-1 Damping System (DS) and Seismic-Force-Resisting System (SRFS) Configurations

elements, or all elements in common with the *seismic-force-resisting system.* Elements common to the *damping system* and the *seismic-force-resisting system* must be designed for combined loads of the two loads of the two systems.

The *seismic-force-resisting system* may be thought of as a collection of lateral-force resisting elements of the *structure* if the *damping system* was not functional (e.g., *damping devices* were disconnected). This system is required to be designed for not less than 75 percent of the *base shear* of a conventional *structure* (not less than 100 percent, if the *structure* is highly irregular), using an *R* factor as defined in Table 5.2.2. This system provides both a safety net against damping system malfunction as well as the stiffness and strength necessary for the balanced lateral displacement of the damped *structure.*

The appendix requires the *damping system* to be designed for the actual (non-reduced) earthquake forces (e.g., peak force occurring in *damping devices*). For certain elements of the *damping system*, other than *damping devices*, limited yielding is permitted provided such behavior does not affect *damping system* function or exceed the amount permitted by the *Provisions* for elements of conventional *structures*.

The *damping devices* include damper units and all pins, bolts, gusset plates, brace extensions and other *components* required to connect damping devices to other elements of the structure. Following the same approach as that used for design of seismic isolators, *damping devices* must be designed for *maximum earthquake* displacements, velocities and forces. Likewise, prototype damper units must be fully tested to demonstrate adequacy for *maximum earthquake* loads and to establish design properties (e.g., effective damping).

Effective Damping

The appendix reduces the response of a *structure* with a *damping system* by the damping coefficient, B, based on the effective damping, β, of the mode of interest. This is the same approach as that used by the *Provisions* for isolated structures. Values of the B coefficient recommended for design of damped *structures* are same as those in the *Provisions* for isolated *structures* at damping levels up to 30 percent, but now extend to higher damping levels based on a recent MCEER study by Constantinou, et al. Like isolation, effective damping of the fundamental-mode of a damped structure is based on the nonlinear force-deflection properties of the *structure*. For use with linear analysis methods, nonlinear properties of the *structure* are inferred from overstrength, Ω_O, and other terms of the *Provisions*. For nonlinear analysis methods, properties of the *structure* would be based on explicit modeling of the post-yield behavior of elements.

Figure CA13-2 illustrates reduction in design earthquake response of the fundamental mode due to effective damping coefficient, B_{1D}. The capacity curve is a plot of the nonlinear behavior of the fundamental mode in spectral acceleration/displacement coordinates. Damping reduction is applied at the effective (secant stiffness) period of the fundamental mode of vibration.

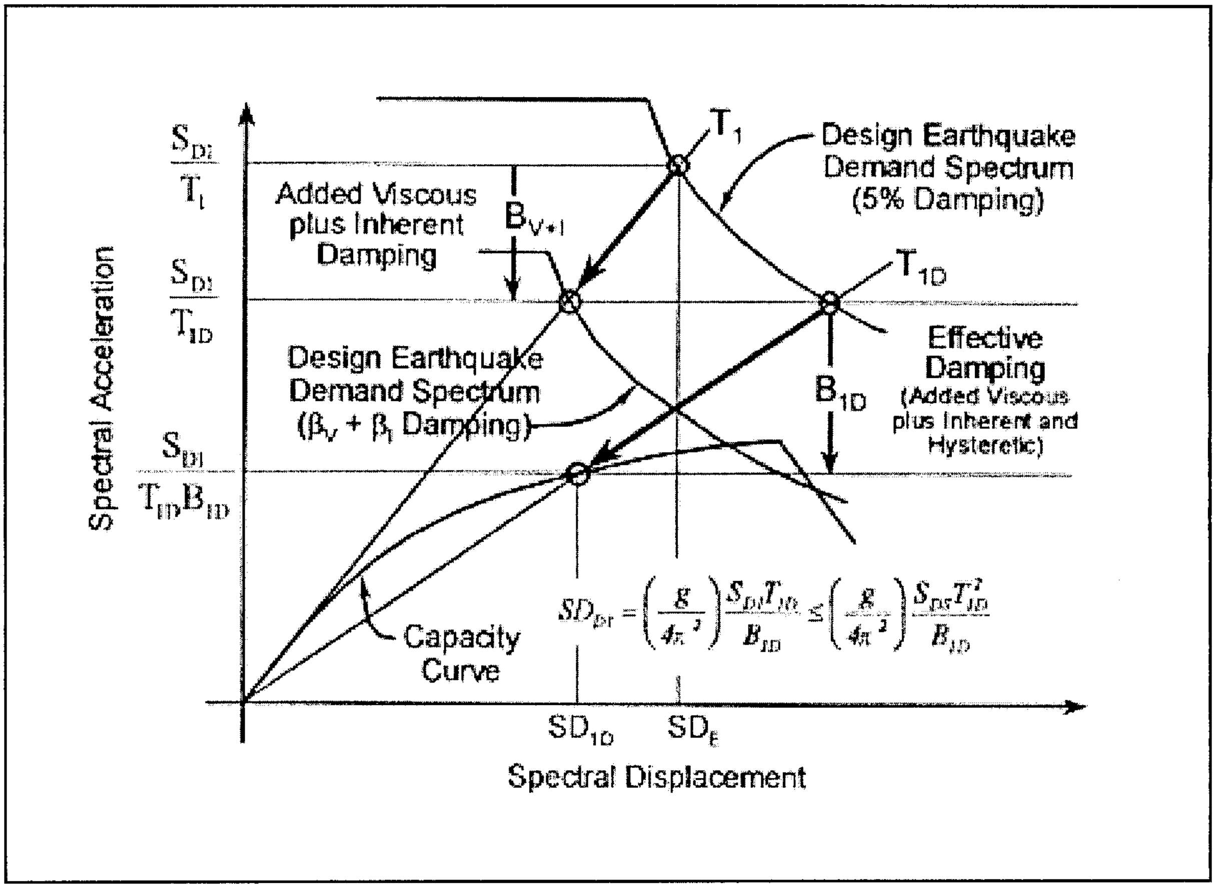


FIGURE C13A-2. Effective Damping Reduction of Design Demand

In general, effective damping is a combination of three components:

1. Inherent Damping β Inherent damping of *structure* at or just below yield, excluding added viscous damping (e.g., typically assumed to be 5 percent of critical for structural systems without dampers).
2. Hysteretic Damping β Post-yield hysteretic damping of the *seismic-force-resisting system* at the amplitude of interest (i.e., taken as 0 percent of critical at or below yield).
3. Added Viscous Damping β Viscous component of the damping system (i.e., taken as 0 percent for hysteretic or friction-based damping systems).

Both hysteretic damping and the effects of added viscous damping are amplitude dependent and the relative contributions to total effective damping changes with the amount of post-yield response of the *structure*. For example, adding dampers to a structure decreases post-yield displacement of the *structure* and hence decreases the amount of hysteretic damping dissipated by the *seismic-force-resisting system*. If the displacements were reduced to the point of yield, the hysteretic component of effective damping would be zero and the effective damping would be equal to inherent damping plus added viscous damping. If there were no *damping system* (i.e., conventional *structure*), then effective damping would simply be equal to inherent damping (e.g., typically assumed to be 5 percent of critical for most conventional *structures*).

Design Earthquake Response Linear Analysis Methods

The appendix specifies *design earthquake* displacements, velocities and forces in terms of *design earthquake* spectral acceleration and modal properties. For linear static analysis, response is defined by two modes: (1) the fundamental mode, and (2) the residual mode. The residual mode is a new concept used to approximate the combined effects of higher modes. While typically of secondary importance to inter-story drift, higher modes can be a significant contributor to inter-story velocity and hence are important for design of *velocity-dependent damping devices*. For response spectrum analysis, higher modes are explicitly evaluated.

For either linear static or response spectrum analysis, response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the *structure*. Nonlinear (pushover) properties, expressed in terms of *base shear* and roof displacement, are related to building capacity, expressed in terms of spectral coordinates, using mass participation and other fundamental-mode factors shown in Figure CA13-3. The conversion concepts and factors shown in Figure CA13-3 are the same as those defined in Chapter 9 of *NEHRP Guidelines* (FEMA 273) for seismic rehabilitation of a *structure* with *damping devices*.

When using linear analysis methods, the shape of the fundamental-mode pushover curve is not known and an idealized elasto-plastic shape is assumed, as shown in Figure CA13-4. The idealized pushover curve shares a common point with the actual pushover curve at the *design earthquake* displacement, D_{1D}. The idealized curve permits defining global ductility demand due to the *design earthquake*, μ_D, as the ratio of design displacement, D_{1D}, to the yield displacement, D_Y. This ductility factor is used in the calculation of various design factors and to set limits on the building ductility demand, μ_{max}, that are consistent with conventional building response limits. Design examples using linear analysis methods have been developed and found to compare well with the results of nonlinear time history analysis (Ramirez et al., 2000).

The appendix requires elements of the *damping system* to be designed for actual fundamental-mode *design earthquake* forces corresponding to a *base shear* value of V_Y (except *damping devices* are designed and prototype tested for *maximum earthquake* forces). Elements of the *seismic-force-resisting system* are designed for reduced fundamental-mode *base shear*, V_1, where force reduction is based on system overstrength, Ω_O, conservatively decreased by the ratio, C_d/R, for elastic analysis (when actual pushover strength is not known).

References:

Ramirez, O.M., M.C. Constantinou, C.A. Kircher, A. Whittaker, M. Johnson and J.D. Gomez. 2000. *Development and Evaluation of Simplified Procedures of Analysis and Design for Structures with Passive Energy Dissipation Systems*, Technical Report MCEER-00-0010, Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo, State University of New York, Buffalo, NY.

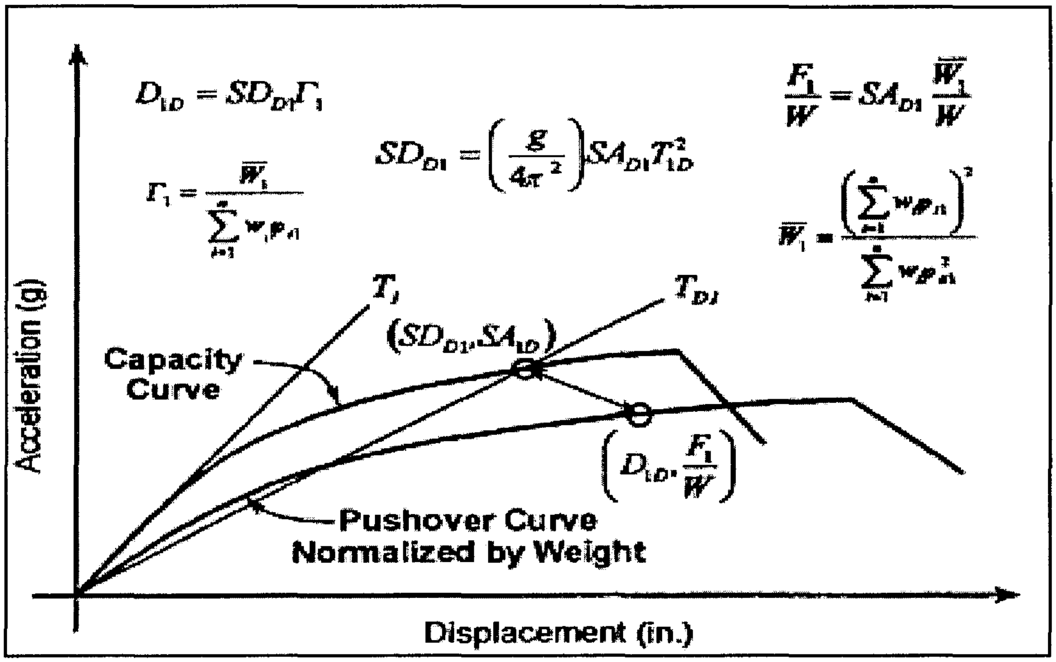


FIGURE C13A-3. Pushover and Capacity Curves

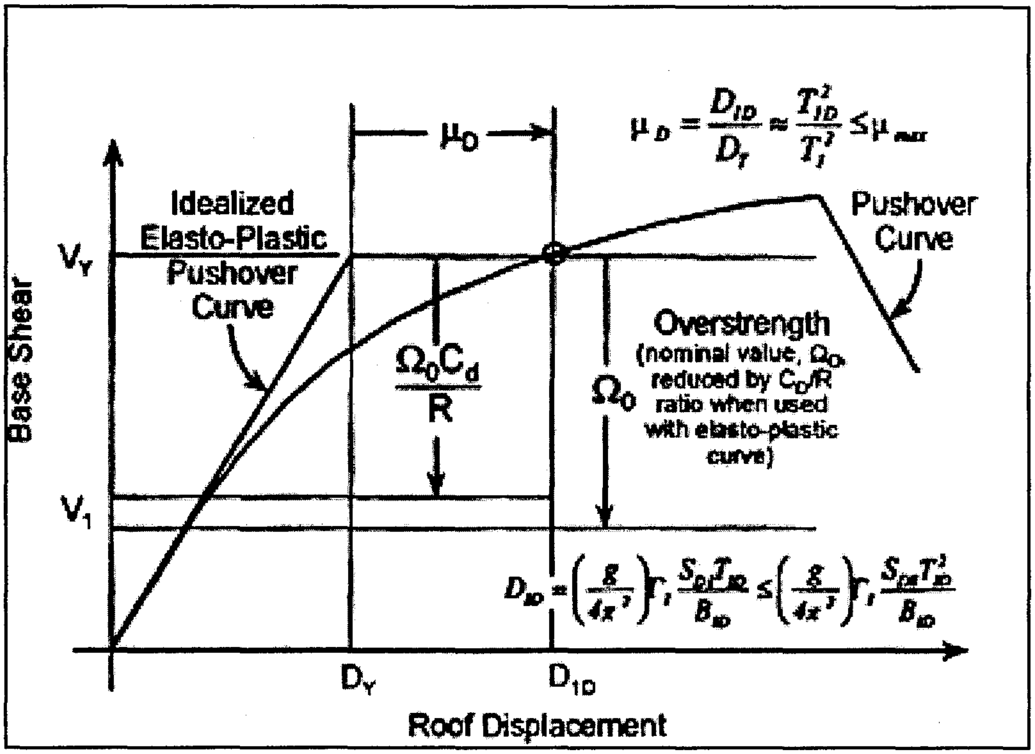


FIGURE C13A-4. Idealized Elasto-Plastic Pushover Curve Used for Linear Analysis

Chapter 14 Commentary

NONBUILDING STRUCTURE DESIGN REQUIREMENTS

14.1 GENERAL:

14.1.1 Scope: Requirements concerning nonbuilding structures were originally added to the 1994 *Provisions* by the 1991-94 *Provisions* Update Committee (PUC) at the request of the BSSC Board of Direction to provide building officials with needed guidance. In recognition of the complexity, nuances and importance of nonbuilding structures, the BSSC Board established 1994-97 PUC Technical Subcommittee 13 (TS13), Nonbuilding Structures, in 1995. The duties of TS13 were to review the 1994 *Provisions* and *Commentary* and recommend changes for the 1997 Edition. The subcommittee was composed of individuals possessing considerable expertise concerning various specialized nonbuilding structures and representing a wide variety of industries concerned with nonbuilding structures.

Building codes traditionally have been perceived as minimum standards of care for the design of nonbuilding structures and building code compliance of these structures is required by building officials in many jurisdictions. However, requirements in the industry standards are often at odds with building code requirements. In some cases, the industry standards need to be altered while in other cases the building codes need to be modified. Registered design professionals are not always aware of the numerous accepted standards within an industry or if the accepted standards are adequate. It is hoped that the 1997 *Provisions* requirements for nonbuilding *structures* appropriately bridge the gap between building code and existing industry standards.

One of TS13's goals was to review and list appropriate industry standards to serve as a resource. These standards had to be included in the appendix. The subcommittee also has attempted to provide an appropriate link so that the accepted industry standards can be used with the seismic ground motions established in the *Provisions*. It should be noted that some nonbuilding structures are very similar to a building and can be designed employing sections of the *Provisions* directly whereas other nonbuilding structures require special analysis unique to the particular type of nonbuilding structure.

The ultimate goal of TS13 was to provide guidance to develop requirements consistent with the intent of the *Provisions* while allowing the use of accepted industry standards. Some of the referenced standards are consensus documents while others are not.

One good example of the dilemma posed by the conflicts between the *Provisions* and accepted design practice for nonbuilding structures are steel multilegged water towers. Historically, such towers have performed well when properly designed per American Water Works Association (AWWA) standards, but these standards differ from the *Provisions* because tension-only rods are required and the connection forces are not amplified. However, industry practice requires upset rods that are preloaded at the time of installation, and the towers tend to perform well in earthquake areas.

In an effort to provide the appropriate interface between the *Provision's* requirements for building structures, nonstructural *components*, and nonbuilding structures; TS13 recommended that nonbuilding structure requirements be placed in a separate chapter. The PUC agreed with this change. The 1997 *Provisions* Chapter 14 now provides registered design professionals responsible for designing nonbuilding structures with a single point of reference.

Note that building structures, vehicular and railroad bridges, nuclear power plants, and dams are excluded from the scope of the nonbuilding structure requirements. The excluded structures are covered either by other sections of the *Provisions* or by other well established design criteria (vehicular and railroad bridges, nuclear power plants, and dams).

14.2 REFERENCES:

American Concrete Institute, (ACI):

ANSI/ACI 349-90 *Code Requirements for Nuclear Safety Related Structures* - Appendix B, 1990. (ACI 349)

ACI 350-99, *Environmental Concrete Concrete Structures*, 1999. (ACI 350)

ACI 307, Standard Practice for the Design and Construction of Cast-In-Place Reinforced Concrete Chimneys, 1995. (ACI 307)

ASCE American Society of Civil Engineers (ASCE), New York:

Petrochemical Energy Committee Task Report, "Guidelines for Seismic Evaluation and Design of Petrochemical Facilities", ASCE publication, 1997. (ASCE Guidelines for Seismic Evaluation and Design of Petrochemical Facilities)

Guidelines for the Seismic Design of Oil and Gas Pipeline Systems, New York, NY, 1984 (ASCE *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*).

Gaylord and Gaylord, Design of Steel Bins for Storage of Bulk Solids, Prentice Hall, 1984. (Gaylord and Gaylord 1984)

Housner, G.W. *Earthquake Pressures in Fluid Containers*, California Institute of Technology (Housner 1954).

Miller, C. D., Meier, S. W., Czaska, W. J., *Effects of Internal Pressure on Axial Compressive Strength of Cylinders and Cones,* Structural Stability Research Council Annual Technical Meeting, June 1997. (Miller 1997)

NFPA National Fire Protection Association

Standard, ANSI/NFPA 30-1996, *Flammable and Combustible Liquids Code*, 1996. (NFPA 30)

Standard, ANSI/NFPA 58-1995, *Storage and Handling of Liquefied Petroleum Gas.* (NFPA 58)

Standard, ANSI/NFPA 59-1998, *Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants*. (NFPA 59)

Standard, ANSI/NFPA 59A-1996, *Production, Storage and Handling of Liquefied Natural Gas (LNG)*. (NFPA 59A)

RMI Rack Manufacturers Institute

Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, 1997. (RMI)

Troitsky, M.S., Tubular Steel Structures by, 1990. (Troitsky 1990)

Wozniak, R. S. and Mitchell, W. W, *Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks*, 1978 Proceedings -- Refining Dept, Vol 57, American Petroleum Institute, Washington, D.C.,May 9, 1978. (Wozniak 1978)

Zick, L.P., *Stresses in Large Horizontal Cylindrical Pressure Vessels on Two Saddle Supports*, Steel Plate Engineering Data, Vol 1and2, American Iron and Steel Institute, Dec 1992. (Zick 1992)

14.4 NONBUILDING STRUCTURES SUPPORTED BY OTHER STRUCTURES: This section has been developed to provide an appropriate link between the requirements for nonbuilding structures and those for inclusion in the rest of the *Provisions*, especially the requirements for architectural, mechanical, and electrical *components*.

14.5 STRUCTURAL DESIGN REQUIREMENTS:

14.5.1 Design Basis: The subcommittee wanted to employ the new seismic ground motion maps and the new methodology for establishing seismic design and detailing contained in the 1997 *Provisions*.

14.5.1.1 Seismic Factors: Table 14.2.1.1 has been formulated to be consistent with the *Provisions*. The values listed here are generally lower than the values for buildings. Lower values are assigned in recognition of the structural performance of nonbuilding structures as opposed to building structures. Nonbuilding structures tend to be lightly damped, less redundant, and more given to performance failure when the structure exhibits nonlinear performance.

14.5.1.2 Importance Factors and Seismic Use Groups Classifications: The Importance Factors and Seismic Use Group classifications assigned nonbuilding structures vary from those assigned building structures. Buildings are designed to protect occupants inside the structure whereas nonbuilding structures are not normally "occupied" in the same sense as buildings, but need to be designed in a special manner because they pose a different sort of risk in regard to public safety (i.e., they may contain very hazardous compounds or be essential *components* in critical lifeline systems). For example, tanks and vessels may contain materials that are essential for lifeline functions following a seismic event (i.e., fire fighting, potable water), potentially harmful or hazardous to the environment or general health of the public, biologically lethal or toxic, or explosive or flammable (threat of consequential or secondary damage).

If not covered by the authority having jurisdiction, Table 14.5.1.2 may be used to select the importance factor *(I)*. The value shall be determined by the largest value from the approved Standards, or largest value selected from Table 14.2.1.2. It should be noted that an entire facility need not be restricted to use only one single value of important factor. For further details, refer to

ASCE *Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (*ASCE, 1997). Also, Use of Secondary Containment System, when designed in accordance with an acceptable National Standards, could be considered as an effective means to contain hazardous substance hence reduce the level of H selection.

The specific definition of material hazard and what constitutes a hazard is currently being developed in the 2000 International Building Code process.. The hazards will be predicated on the quantity and type of hazardous material.

The importance factor is not intended for use in making economic evaluations regarding the level of damage, probabilities of occurrence, or cost to repair the structure. These economic decisions should be made by the owner and other interested parties (insurers, financiers, etc). Nor it is intended for use for other purposes other than that defined in this provision. This include use of higher important factor in order to compensate the use of Site Specific Response Spectra.

Following are examples demonstrating how this table may be applied:

Example 1:

A water storage tank used to provide pressurized potable water for a process within a chemical plant where the tank is located away from personnel working within the facility.

TABLE 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

Importance Factor	$I = 1.0$	$I = 1.25$	$I = 1.5$
Seismic Use Group	**I**	**II**	**III**
Hazard	H - I	H - II	H - III
Function	F - I	F - II	F - III

Address each of the issues implied in the matrix:

Seismic Use Group — Neither the structure nor the contents are critical, therefore use Seismic Use Group I.

Hazard — The contents are not hazardous, therefore use H - I.

Function — The water storage tank is not a designated ancillary structure for post–earthquake recovery, nor serves as emergency back-up facilities for a Seismic Use Group III structure, therefor use F - I.

This tank has an importance factor of 1.0.

Example 2:

A steel storage rack is located in a retail store in which the customers have direct access to the aisles. Merchandise is stored on the upper racks. The rack is supported from a slab on grade.

TABLE 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

Importance Factor	***I*** **= 1.0**	***I*** **= 1.25**	***I*** **= 1.5**
Seismic Use Group	**I**	**II**	**III**
Hazard	H - I	H - II	H - III
Function	F - I	F - II	F - III

Address each of the issues in the matrix:

Seismic Use Group — Neither the structure nor the contents are critical, therefor use Seismic Use Group I.

Hazard — The contents are not hazardous, however its use could cause a substantial public hazard during earthquake, – subject to local Authority's jurisdiction it is H-II.

Function — The storage rack is not used for earthquake recovery, nor is it required for emergency back-up, therefore use F - I.

Within the steel storage rack section in the *Provisions* there exists a link back to Sec. 6.9 and to Sec. 6.1.5 requiring an I_p or I of 1.5.

Use an importance factor of 1.5 for this structure.

Example 3:

A water tank is located within an office building complex to supply the fire sprinkler system.

TABLE 14.5.1.2 Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

Importance Factor	***I* = 1.0**	***I* = 1.25**	***I* = 1.5**
Seismic Use Group	**I**	**II**	**III**
Hazard	H - I	H - II	H - III
Function	F - I	F - II	F - III

Address each of the issues in the matrix:

Seismic Use Group — The office building is Seismic Use Group I.

Hazard — The content and its use are not hazardous to the public, therefore use H - I.

Function — The water tank is required to provide water for fire fighting, however since the Building is not a Seismic Use Group III structures, the water is not used for post earthquake recovery, nor is it required for emergency back-up, therefor use F - I.

Use an importance factor of 1.0 for this water structure.

Example 4:

A petro-chemical storage tank is to be constructed within a refinery tank farm near a populated City neighborhood. Impoundment dike is provided to control liquid spills.

Table 14.5.1.2
Importance Factor (*I*) and Seismic Use Group Classification for Nonbuilding Structures

Importance Factor	***I* = 1.0**	***I* = 1.25**	***I* = 1.5**
Seismic Use Group	**I**	**II**	**III**
Hazard	H - I	H - II	H - III
Function	F - I	F - II	F - III

Address each of the issues in the matrix.:

Seismic Use Group — The LNG tank is Seismic Use Group III.

Hazard — The contents constitute a sufficient quantities of high explosive and is near a city neighborhood, despite the diking, it is considered hazardous to the public under earthquake, therefore use H - III.

Function — The tank is not required to provide post-earthquake recovery, nor is used for emergency back-up for Seismic Use Group III structures therefore use F - I.

Use an importance factor of 1.5 for this structure.

14.5.2 Rigid Nonbuilding Structures: The equation included in the 1994 *Provisions* did not agree with the formulas contained in the 1994 *Uniform Building Code* (*UBC*). The Seismic Design Procedure Group recommended using the S_{DS} factor and eliminating the C_a factor. The appropriate changes are incorporated in the 1997 *Provisions*.

14.5.4 Fundamental Period: The rational methods for period calculation contained in the *Provisions* were developed for building structures. If the nonbuilding structure has dynamic characteristics similar to a building, the difference in period is insignificant. If the nonbuilding structure is not similar to a building structure, other techniques for period calculation will be required. Some of the references in for specific types of nonbuilding structures may contain more accurate methods for period determination.

14.6 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS: This general class of nonbuilding structures exhibits behavior similar to that of building structure; however, function and performance are different. The *Provisions* were used as the primary basis for design with industry-driven exceptions, modifications, and additions.

14.6.2 Pipe Racks: Free standing pipe racks supported at or below grade with framing systems that are similar in configuration to building systems, should be designed to meet the force requirements of Sec. 5.4. Single column pipe racks that resist lateral loads should be designed as inverted pendulums. See ASCE "Guidelines for the Seismic Evaluation and design of Petrochemical Facilities (1997).

14.6.3 Steel Storage Racks:

This section is intended to assure comparable results from the use of the RMI Specification, the NEHRP *Provisions*, and the IBC code approaches to rack structural design and to distinguish between the methods employed to design storage racks supported at grade (as treated in Sec. 14.3.3 Steel Storage Racks, Nonbuilding Structures) from those supported above grade (as treated in Sect. 6.1 Architectural, Mechanical, and Electrical Components Seismic Design Requirements). This will help clarify and coordinate the multiple references to rack structures in these *Provisions* and the different means by which rack structures are analyzed and designed.

The RMI for many years has been working with the various committees of the model code organizations and of the Building Seismic Safety Council and its Technical Sub-Committees to create seismic design provisions particularly applicable to steel storage rack structures. The new 1997 RMI Specification is seen to be in concert with the needs, provisions, and design intent of the building codes and those who use and promulgate them, as well as those who engineer, manufacture, install, operate, use and maintain rack structures. The new RMI Specification, now including detailed seismic provisions, is seen to be self-sufficient. The 1997 Edition of the RMI Specification is presently undergoing the ANSI canvassing process.

The changes proposed here are compatible and coordinated with the changes recently approved, in March 1999, by the IBC Structural Committee for inclusion in the IBC 2000.

14.6.4 Electrical Power Generating Facilities: Electrical power plants closely resemble building structures, and their performance in seismic events has been good. For reasons of mechanical performance, lateral drift of the structure must be limited. The lateral bracing system of choice has been the concentrically braced frame. The height limits on braced frames in particular can be an encumbrance to the design of large power generation facilities. For this reason, the exception to height limits in Sec. 14.5.1 was required.

14.6.6 Piers and Wharves: Although previous editions of the *Provisions* did not include a specific section on piers and wharves, the inclusion of these structures was deemed necessary to properly account for the effect of hydrodynamic and liquefaction effects unique to these types of structures.

14.7 NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS: This general class of nonbuilding structures exhibits behavior markedly different from that of building structures. Most of these types of structures have industry standards that address their unique structural performance and behavior. The new elements of the *1997 Provisions* regarding ground motion required that a prudent link to the industry standards be developed.

14.7.1 General:

14.7.2 Earth Retaining Structures: In order to properly develop and implement methodologies for the design of earth retaining structures it is essential to know and understand the nature of the applied loads. Concerns have been raised on how to design nonyielding walls and yielding walls for bending, overturning, sliding, etc., taking into account the varying soil types, importance, and site seismicity. See Sec. 7.5.1 in the *Commentary*.

14.7.3 Tanks and Vessels:

14.7.3.1 General: Methods of seismic design of tanks, currently adopted by a number of industry standards have evolved from earlier analytical work by Jacobsen, Housner, Veletsos, Haroun, and others. The procedures used to design flat bottom storage tanks and liquid containers is based on the work of Housner and Wozniak and Mitchell. The standards for tanks and vessels have specific requirements to safeguard against catastrophic failure of the primary structure based on observed behavior in seismic events since the 1930s. Other methods of analysis using flexible shell models have been proposed but are presently beyond the scope of these Provisions

These methods entail three fundamental steps:

I. The dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to a ground acceleration, the lower portion of the contained liquid, identified as the impulsive component of mass W_I, acts as if it were a solid mass rigidly attached to the tank wall. As this mass accelerates, it exerts a horizontal force, P_I, against the wall that is directly proportional to the maximum acceleration of the tank base. This force is superimposed on the inertia force of the accelerating wall itself, P_w. Under the influence of the same ground acceleration, the upper portion of the contained liquid responds as if it were a solid liquid mass flexibly attached to the tank wall. This portion, which oscillates at its own natural frequency, is identified as the *convective component* W_c, and exerts a force P_c on the wall. The convective component oscillations are characterized by the phenomenon of sloshing whereby the liquid surface rises above the static level on one side of the tank, and drops below that level on the other.

II. The determination of the frequency of vibration, w_I, of the tank structure and the impulsive component; and the natural frequency of oscillation (sloshing), w_c, of the convective component.

III. The selection of the design response spectrum. The response spectrum may be site-specific; or it may be constructed deterministically on the basis of seismic coefficients given in national codes and standards. Once the design response spectrum is constructed, the spectral accelerations corresponding to w_I and w_c are obtained and are used to calculate the dynamic forces P_I, P_w, and P_c.

Detailed guidelines for the seismic design of circular tanks, incorporating these concepts to varying degrees, have been the province of at least four industry standards: AWWA D100 for welded steel tanks (since 1964); API 650 for petroleum storage tanks; AWWA D110 for prestressed, wire-wrapped tanks (since 1986); and AWWA D115 for prestressed concrete tanks stressed with tendons (since 1995). In addition, API 650 and API 620, contain provisions for petroleum, petrochemical and cryogenic storage tanks. The detail and rigor of analysis employed by these standards have evolved from a semi-static approach in the early editions to a more rigorous approach at the present reflecting the need to factor in the dynamic properties of these structures.

The requirements in Sec 14.7.3 are intended to link the latest procedures for determining design level seismic loads with the allowable stress design procedures based on the methods in these Provisions. These requirements, which in many cases identify specific substitutions to be made in the design equations of the national standards, will assist users of the *Provisions* in making consistent interpretations.

More recently, ACI Committee 350 has drafted a document, ACI 350.3, titled *"ACI Practice for the Seismic Design of Liquid-Containing Structures"*. This document, which covers all types of concrete tanks (prestressed and non-prestressed, circular and rectilinear), is currently being revised to conform with the seismic risk guidelines of NEHRP 1997 and IBC 2000. This ACI *"Practice"* will serve as a practical, "how-to" - and yet rigorous - guide to supplement Chapter 21 (*"Special Provisions for Seismic Design"*) of ACI 350.

14.7.3.2 Design Basis: Two important tasks of TS-13 are to (a) partially expand the Provision's coverage of nonbuilding structures; and (b) provide comprehensive cross-references to all the applicable industry standards. This endeavor will hopefully bring about a standardization and consistency of design practices for the benefit of both the practicing engineer and the public at large.

In the case of the seismic design of nonbuilding structures, standardization will probably necessitate certain adjustments on the part of current industry standards to minimize existing inconsistencies among them. At the same time, however, this process must take cognizance of the fact that structures designed and built over the years in accordance with these standards have performed well in earthquakes of varying severity.

The most important inconsistencies among current standards that need to be addressed in any standardization/update process relate primarily to differences in the derivation of the terms that make up the traditional base shear equation :

$$V = \frac{ZIS}{R_w} CW$$

An examination of those terms as currently used in the different references reveals the following:

- ZS: The "Seismic zone coefficient" Z has been rather consistent among all these standards by virtue of the fact that it has traditionally been obtained from the seismic zone designations and maps of the national building codes.

  On the other hand, "Soil Profile Coefficient" S does vary from one standard to another. In some standards these two terms are combined.

- I: Importance Factor I has also varied from one standard to another but this variation is unavoidable and understandable owing to the multitude of uses and degrees of importance of liquid-containing structures.

- C: Coefficient C represents the dynamic amplification factor that defines the shape of the design response spectrum for any given maximum ground acceleration. Since coefficient C is primarily a function of the frequency of vibration, inconsistencies in its derivation from one standard to another stem from at least two sources: Differences in the equations for the determination of the natural frequency of vibration; and differences in the equation for the coefficient C itself. For example, for the shell/impulsive liquid component of lateral force, the steel tank standards use a constant design spectral acceleration (namely, a constant C) that is independent of the "impulsive" period T. In addition, the value of C will vary depending on the damping ratio assumed for the vibrating structure (2 percent - 7 percent).

Where a site-specific response spectrum is available, calculation of coefficient C is not necessary – except in the case of the convective component (coefficient C_c) which is assumed to oscillate with 0.5 percent of critical damping, and whose period of oscillation is usually high (>2.5 seconds). Since site-specific spectra are usually constructed for high damping values (3 percent - 7 percent); and since the site-specific spectral profile may not be well-defined in the high-period range, an equation for C_c applicable to 0.5 percent damping ratio is necessary in order to calculate the convective component of the seismic force.

R: The Response Modification Factor R_w is perhaps the most difficult to quantify, for a number of reasons. While R_w is a compound coefficient that is supposed to reflect the ductility, energy-dissipating capacity, and structural redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In NEHRP 1997 and IBC 2000, the base shear equation for most structures has been reduced to $V = C_s W$, where the Seismic Response Coefficient C_s replaces the product $\frac{ZSC}{R_w}$. C_s is determined from the Design Spectral Response Accelerations S_{DS} or S_{D1} (at short periods, or at 1 second period respectively) which, in turn, are obtained from the mapped MCE (Maximum Considered Earthquake) spectral accelerations S_s and S_1 obtained from the new seismic maps. As in the case of the prevailing industry standards, where a site-specific response spectrum is available, C_s is replaced by the actual spectral values of that spectrum.

As part of its task, TS-13 has introduced a number of provisions, each designed to provide a means of properly applying the design criteria of a particular industry standard with the latest NEHRP practices. These provisions are outlined below and are identified with particular types of liquid-containing structures and the corresponding standards. Underlying all these provisions is the understanding that the calculation of the periods of vibration of the impulsive and convective *components* is left up to the industrial standards. Defining the detailed resistance and allowable stresses of the structural elements for each industrial structure has also been left to the approved standard except in instances where additional information has led to additional requirements.

14.7.3.3 Strength and Ductility: As is the case for building structures, ductility and redundancy in the lateral support systems for tanks and vessels are desirable and necessary for good seismic performance. Tanks and vessels are not highly redundant structural systems and, therefore, ductile materials and well-designed connection details are needed to increase the capacity of the vessel to absorb more energy without failure. The critical performance of many tanks and vessels is governed by shell stability requirements rather than by yielding of the structural elements. For example, contrary to building structures, ductile stretching of the anchor bolts is a desirable energy absorption component when tanks and vessels are anchored. The performance of cross-braced towers is highly dependent on the ability of the horizontal compression struts and connection details to fully develop the tension yielding in the rods. In such cases, it is also important that the rods stretch and do not fail prematurely in the threaded portion of the connection, or the connection of the rod to the column fail prior to yielding of the rod.

14.7.3.4 Flexibility of Piping Attachments: The performance of piping connections under seismic deformations is one of the primary weaknesses observed in recent seismic events. Tank leakage and damage occurs when the piping connections cannot accommodate the movements the tank experiences during the a seismic event. Contrary to the design methods used by many piping designers, which impart mechanical loading to the tank shell, piping systems in seismic areas should be designed in such a manner as to impose negligible mechanical loads on the tank connection for the values shown in Table 14.4.3.1.2.

In addition, interconnected equipment, walkways, and bridging between multiple tanks must be designed to resist the loads and displacements imposed by seismic forces. Unless multiple tanks are founded on a single rigid foundation, walkways, piping, bridges and other connecting structures must be designed to allow for the calculated differential movements between connected structures due to seismic loading assuming the tanks and vessels are out of phase

14.7.3.5 Anchorage: Many steel tanks can be designed without anchors by using the annular plate procedures given in the national standards. Tanks that must be anchored because of overturning potential could be susceptible to shell tearing if not properly designed. Ideally, the proper anchorage design will provide both a shell attachment and embedment detail that will yield the bolt without tearing the shell or pulling the bolt out the foundation. Properly designed anchored tanks retain greater reserve strength to resist seismic overload than unanchored tanks.

Premature failure of anchor bolts has been observed when the bolt and attachment are not properly aligned (i.e the anchor nut or washer does not bear evenly on the attachment). Additional bending stresses in threaded areas may cause the anchor to fail before yielding

14.7.3.6 Ground-Supported Storage Tanks for Liquids:

14.7.3.6.1 General: The response of ground storage tanks to earthquakes is well documented by Housner, Mitchell and Wozniak, Veletsos, and others. Unlike building structures, the structural response is strongly influenced by the fluid-structure interaction. Fluid-structure interaction forces are categorized as sloshing (convective mass) and rigid (impulsive mass) forces. The proportion of these forces depends on the geometry (height to diameter ratio) of the tank. API 650, API 620, AWWA D100, AWWA D110, AWWA D115, and ACI 350.3provide the necessary data to determine the relative masses and moments for each of these contributions.

The Provisions stipulate that these structures shall be designed in accordance with the prevailing approved industry standards, with the exception of the height of the sloshing wave, d_s, which is defined by equation (14.7.3.7.1) of these Provisions.

$$\delta_s = 0.5DIS_{ac}$$

This equation utilizes a spectral response coefficient $S_{ac} = \frac{1.5 S_{D1}}{T_c}$ for $T_c < 4.0$ sec., and

$S_{ac} = \frac{6 S_{D1}}{T_c^{\,2}}$ for $T_c > 4.0$ sec. The first definition of S_a represents the constant-velocity region of the response spectra and the second the constant-displacement region of the response spectrum at 0.5 percent damping. In practical terms, the latter is the most commonly used definition since most tanks have a fundamental period of liquid oscillation (sloshing wave period) greater than 2.5 sec., and, most commonly, greater than 4.0 sec.

Small diameter tanks and vessels are more susceptible to overturning and vertical buckling. As a general rule, the greater the ratio of H/D, the lower the resistance is to vertical buckling. When H/D > 2, the overturning begins to approach "rigid mass" behavior (the sloshing mass is small). Large diameter tanks may be governed by additional hydrodynamic hoop stresses in the middle regions of the shell.

The impulsive period (the natural period of the tank *components* and the impulsive component of the liquid) is typically in the 0.25 to 0.6 second range. Many methods are available for calculating the impulsive period. The Veletsos flexible shell method is commonly used by many tank designers. (For example, see "Seismic Effects in Flexible Liquid Storage Tanks" A.S. Veletsos).

14.7.3.6.1.1 Distribution of Hydrodynamic and Inertia Forces: Most of the methods contained in the industry standards for tanks define reaction loads at the base of the shell and foundation interface. Many of the standards do not give specific guidance for determining the distribution of the loads on the shell as a function of height. The design professional may find the additional information contained in ACI 350.3 helpful.

The overturning moment at the base of the shell is defined in the industry standards is only the portion of the moment that is transferred to the shell. It is important the design professional realize that this total overturning moment must also include the variation in bottom pressure. This is important when designing pile caps, slabs or other support elements that must resist the total overturning moment. See Wozniak 1978 or TID 7024 for further information.

14.7.3.6.1.2 Freeboard: Performance of ground storage tanks in past earthquakes has indicated that sloshing of the contents can cause leakage and damage to the roof and internal *components*. While the effect of sloshing often involves only the cost and inconvenience of making repairs, not catastrophic failure, even this limited damage can be prevented or significantly mitigated when the following aspects are considered:

1. Effective masses and hydro-dynamic forces in the container
2. Impulsive and pressure loads.
   a. Sloshing zone (i.e. the upper shell and edge of roof system).
   b. Internal supports (roof support columns, tray-supports, etc.).
   c. Equipment (distribution rings, access tubes, pump wells, risers, etc.).
3. Freeboard (depends on the sloshing wave height).

A minimum freeboard of $0.7\delta_s$ is recommended for economic considerations but not required.

Tanks and vessels storing biologically or environmentally benign materials do not typically require freeboard to protect the public health and safety. However, providing freeboard in areas of frequent seismic occurrence for vessels normally operated at or near top capacity may lessen damage (and the cost of subsequent repairs) to the roof and upper container.

The estimate given in the *Provision* Sec 14.7.3.7.1.2 is based on a median response spectrum rather than on the one standard deviation response spectra found in TID 7024. It is also based on the seismic design event as defined by the *Provisions*. Estimates for the sloshing height contained in national standards are based on the one standard deviation spectra applied at a working stress level. Users of the *Provisions* may estimate slosh heights different from those recommended in the national standards.

14.7.3.6.1.5 Sliding Resistance: Steel ground-supported tanks full of product have not been found to slide off foundations. A few unanchored, empty tanks have moved laterally during earthquake. In most cases, these tanks may be returned to their proper locations. Resistance to sliding is obtained from the frictional resistance between the steel bottom and the sand cushion on which bottoms are placed. Because tank bottoms usually are crowned upward toward the tank center and are constructed of overlapping fillet welded individual steel plates (resulting in a rough bottom), it is reasonably conservative to take the ultimate coefficient of friction as 0.70 (U.S. Nuclear Regulatory Commission, 1989, pg A-50) and, therefore, a value of tan 30° (0.577) is used. The vertical weight of the tank and contents reduced by the component of vertical acceleration provides the net vertical load. An orthogonal combination of vertical and horizontal seismic forces following the procedure in Sec.5.2 may be used.

14.7.3.6.1.6 Local Shear Transfer: The lateral seismic shear from the roof to the shell and the shell to the base is resisted by a combination of membrane shear and the radial shear in the wall of the tank. For steel tanks, the radial shear is very small and is usually neglected; thus, the shear is assumed to be carried totally by membrane shear. For concrete walls and shell, which have a greater radial shear stiffness, the shear transfer may be shared. The user is referred to Commentary of ACI 350 for further discussion.

14.7.3.6.1.7 Pressure Stability: Internal pressure may increase the critical buckling capacity of a shell. Provisions to include pressure stability in determining the buckling resistance of the shell for overturning loads is included in AWWA D100. Recent testing on conical and cylindrical shells with internal pressure yielded a design methodology for resisting permanent loads in addition to temporary wind and seismic loads. See Miller, et al 1997.

14.7.3.6.1.8 Shell Support: Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and reduce impact on the anchor bolt under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (i.e. fiberboard) that creates a gap between the anchor and the attachment. As the load reverses, the bolt is no longer snug and an impact of the attachment on the anchor can occur. Grout is a structural element and should be installed and inspected as if it is an important part of the vertical and lateral force resisting system.

14.7.3.6.1.9 Repairs, Alterations, and Modifications: During their service life, storage tanks are frequently repaired, modified or relocated. Repairs or often related to corrosion, improper operation, or overload from wind or seismic events. Modifications are made for changes in service, updates to safety equipment for changing regulations, installation of additional process piping connections. It is imperative these repairs and modifications are properly designed and implemented to maintain the structural integrity of the tank or vessel for seismic loads as well as the design operating loads.

The petroleum steel tank industry has developed specific guidelines in API 653 that are statutory requirements in some states. It is the intent of TS 13 that the provisions of API 653 also be applied to other liquid storage tanks (water, wasterwater, chemical, etc) as it relates to repairs, modifications or relocation that effects the pressure boundary or lateral force resisting system of the tank or vessel.

14.7.3.7 Water and Water Treatment Tanks and Vessels:

14.7.3.7.1 Welded Steel: The AWWA design requirements for of ground-supported steel water storage structures is based on an allowable stress method that utilizes an effective mass procedure considering two response modes of the tank and its contents:

1. The high-frequency amplified response to seismic motion of the tank shell, roof, and impulsive mass (portion of liquid content of the tank that moves in unison with the shell) and
2. The low frequency amplified response of the convective mass (portion of the liquid contents in the fundamental sloshing mode).

The two-part AWWA equation incorporates the above modes, appropriate damping, site amplification, allowable stress response modification and zone coefficients. In practice, the typical ground storage tank and impulsive contents will have a natural period, *T*, of 0.1 to 0.3 sec. The sloshing period typically will be greater than 1 sec (usually 3 to 5 sec depending on tank geometry). Thus, the substitution in the *Provisions* uses a short- and long-period response as it applies to the appropriate constituent term in the AWWA equations.

14.7.3.7.2 Bolted Steel: The AWWA Steel Tank Committee is responsible for the content of both the AWWA D100 and D103 and have established equivalent load and design criteria for earthquake design of welded and bolted steel tanks.

14.7.3.8.3 Reinforced and Prestressed Concrete: Given T_I, the natural period of tank shell plus the confined (impulsive) liquid; and T_C (or T_W), the first-mode sloshing wave period (as defined in 14.7.3.7.1 of these Provisions),

(a) For $T_I < T_o$, and $T_I > T_s$, the term $\frac{ZICi}{Ri}$ in the base shear and overturning moment equations of AWWA D110-95 and D115-95; and the term $\frac{ZISC}{R_i}$ in the base shear and overturning moment equations of draft ACI 350.3 are both replaced by $\frac{S_{d}I}{1.4R}$

(b) For $T_o \le T_I \le T_s$, $\frac{ZICi}{Ri}$ and $\frac{ZISC}{R_i}$ are replaced by $\frac{S_{DS}I}{1.4R}$

(c) For all values of T_C, $\frac{ZIC_s}{R_c}$ and $\frac{ZISC_s}{R_c}$ are replaced by $\frac{6S_{D1}I}{T_c^2}$..*or*...$\frac{6S_{DS}I}{T_c^2}T_s$

where T, S_a, S_1, and S_{DS} are defined in Sec. 4.1.2.6, and T_c.

14.7.3.8 Petrochemical and Industrial Tanks and Vessels Storing Liquids

14.7.3.8.1 Welded Steel: The American Petroleum Institute (API) also uses an allowable stress design procedure and the API equation has incorporated an R_w factor into the equations directly.

The most common damage to tanks observed during past earthquakes include:

- Buckling of the tank shell near the base due to excessive axial membrane forces. This buckling damage is usually evident as "elephant foot" buckles a short distance above the base, or as diamond shaped buckles in the lower ring. Buckling of the upper ring has also been observed
- Damage to the roof due to impingment on the underside of the roof of sloshing liquid with insufficient freeboard
- Failure of piping or other attachments that are overly restrained.
- Foundation failures

The performance of floating roofs during earthquakes has been good with damage usually confined to the rim seals, gage poles and ladders. Similarly the performance of open top with top wind girder stiffeners designed per API 650 has been good.

14.7.3.10 Elevated Tanks and Vessels for Liquids and Granular Materials:

14.7.3.10.4 Transfer of Lateral Forces into Support Tower The lateral transfer of load for tanks and vessels siting on grillage or support beams should consider the relative stiffness of the support beams, and the shear transfer at the base of the shell which is not typically uniform around the base of the tank. In addition, when tanks and vessels are supported on discrete points on grillage or beams, it is common for the vertical loads to vary due to settlements or variations in construction. This variation in load should be considered when analyzing the combined vertical and horizontal loads.

14.7.3.10.5 Evaluation of Structures Sensitive to Buckling Failure: Nonbuilding structures that have low or negligible structural redundancy for lateral loads need to be evaluated for a critical level of performance to provide sufficient margin against premature failure. Reserve strength beyond for loads beyond the design loads can be limited. Tanks and vessels supported on shell skirts or pedestals that are governed by buckling are examples of structures that need to evaluated at this critical condition. Such structures include single pedestal water towers, process vessels, and other single member towers.

The additional evaluation is based on a scaled maximum considered earthquake. This critical earthquake acceleration is defined as the design spectral response acceleration, Sa, which includes site factors. The I/R coefficient is taken as 1.0 for this critical check. The structural capacity of the shell is taken as the critical buckling strength (i.e. the factor of safety is 1.0).

Vertical or orthogonal earthquake combination need not be made for this critical evaluation since the probability of critical peak values occurring simultaneously is very low.

14.7.3.10.7 Concrete Pedestal (Composite) Tanks: A composite elevated water-storage tank is a structure comprising a welded steel tank for watertight containment, a single pedestal concrete support structure, foundation, and accessories.

As these structures began in the market place, the design–build firms developed proprietary standards and methods for their structures. The Steel Plate Fabricators Association developed a guideline specification for this style tank in the early 1990's. After debate, an AWWA Committee was formed in 1992 to prepare a standard for composite elevated water tanks. Also in 1992, the American Concrete Institute Committee 371 began work on a recommended practice for the design and construction of concrete-pedestal water towers (ACI 371R), which was first published in 1998. ACI 371R focused on the application of loads to the structure, and on the design and construction aspects of concrete *components* and foundations. Design and construction requirements for the steel tank were included by reference to national standards. The draft AWWA D170 uses applicable portions of ACI 371R and AWWA D100 as resources, and expands upon them to provide a comprehensive design and construction document for composite elevated water tanks.

There is limited experience with the seismic performance of this type of tank compared to the other styles of elevated water storage tanks built in the US for the past several decades. This style of tank was initially marketed and built in Canada and the southwest US (primarily in Texas), primarily in regions of low seismicity. While this style of tank has spread to cover much of the eastern US, none have been located in an area where a significant seismic event has occurred. The design rules in the Provisions are based on present day design procedures and engineering principles used by the design-builders, the ACI 371 recommended practice, and the draft AWWA standard. All of these methods are at present unproven.

14.7.4 Stacks and Chimneys: The design of stacks and chimneys to resist natural hazards is generally governed by wind design considerations. The exceptions to this general rule involve locations with high seismicity, stacks and chimneys with large elevated masses, and stacks and chimneys with unusual geometries. It is prudent to evaluate the effect of seismic loads in all but those areas with the lowest seismicity. Although not specifically required, it is recommended that the special seismic details required elsewhere in the *Provisions* be evaluated for applicability to stacks and chimneys.

Guyed steel stacks and chimneys are generally light weight. As such the design loads due to natural hazards are generally governed by wind. On occasion, large flares or other elevated masses located near the top may require an in-depth seismic analysis. Although Chapter 6, "Multilevel Guyed Stacks" in *Tubular Steel Structures* by M. S. Troitsky does not specifically address seismic loading, it remains an applicable methodology for resolution of seismic forces that can be determined in these*Provisions*.

Appendix to Chapter 14

PREFACE: The following sections were originally intended to be part of the Nonbuilding Structures Chapter of this Commentary. The *Provisions* Update Committee felt that given the complexity of the issues, the varied nature of the resource documents, and the lack of supporting consensus resource documents, time did not allow a sufficient review of the proposed sections required for inclusion into the main body of the chapter.

The Nonbuilding Structures Technical Subcommittee, however, expressed that what is presented herein represents the current industry accepted design practice within the engineering community that specializes in these types of nonbuilding structures.

The *Commentary* sections are included here so that the design community specializing in these nonbuilding structures can have the opportunity to gain a familiarity with the concepts, update their standards, and send comments on this appendix to the BSSC.

It is hoped that the various consensus design standards will be updated to include the design and construction methodology presented in this Appendix. It is also hoped that industry standards that are currently not consensus documents will endeavor to move their standards through the consensus process facilitating building code inclusion.

C14A.1 REFERENCES:

Agrawal P.K. and Kramer J.M., Analysis of Transmission Structures and Substation structures and Equipment for Seismic Loading, Sargent & Lundy Transmission and Substation Conference, December 2, 1976. (Agrawal 1976)

American Society of Civil Engineers (ASCE):
ANSI/ASCE 10-97, *Design of Latticed Transmission Structures*, New York, NY, 1997. (ASCE 10)

ASCE Manual 72, *Tubular Pole Design Standard*, New York, NY, 1991 (ASCE 72).

ASCE Manual 74, *Guidelines for Electrical Transmission Line Structural Loading*, New York, NY, 2000. (ASCE 74).

ASCE 7-95, *Minimum Design Loads for Buildings and Other Structures*, 1995 (ASCE 7).

American Society of Civil Engineers, (ASCE 1997), ASCE Manual 91, *The Design of Guyed Electrical Transmission Structures*, New York, NY, 1997. (ASCE 97)

Substation Structure Design Guide, New York, NY, 2000. (ASCE 2000)

LI, H-N., Wang, S., Lu, M., and Wang, Q., Aseismic Calculations for Transmission Towers, ASCE Technical Council on Lifeline Earthquake Engineering, Monograph No. 4, August, 1991. (ASCE Li Monograph 4)

Steinhardt, Otto W., Low Cost Seismic Strengthening of Power Systems, Journal of The Technical Councils of ASCE, American Society of Civil Engineers, April 1981. (ASCE Steinhardt 1981)

Ameri, G. G. and McClure, G., Seismic Response to Tall Guyed Telecommunication Towers, Paper No. 1982, Eleventh World Conference on Earthquake Engineering, Elsevier Science Ltd., 1996. (Ameri 1996)

Australian Standards:
Australian Standard 3995, Standard Design of Steel Lattice Towers and Masts, 1994. (AS 3995)

Canadian Standards Association (CSA):
Antennas, Towers, and Masts, 1994, (AS 3995)

Earthquake Engineering Research Institute (EERI):
Li, H-N., Suarez, and Singh M.P., Seismic Effects on High-Voltage Transmission Tower and Cable Systems, EERI Conference, Chicago 1994. (EERI Seismic Effects 1994)

Federal Emergency Management Agency (FEMA):
Earthquake Resistant Construction of Electric Transmission and Telecommunication Facilities Serving the Federal Government Report, FEMA-202, Federal Emergency Management Agency, September 1990. (FEMA 202)

Galvez, C. A., and McClure, G., A Simplified Method for Aseismic Design of Self-Supporting Latticed Telecommunication Towers, Seventh Canadian Conference on Earthquake Engineering, Montreal, 1995. (Galvez 1995)

Institute of Electrical and Electronics Engineers (IEEE):
National Electrical Safety Code, ANSI C2, New Jersey, 1997. (NESC)

IEEE Standard 693, Recommended Practices for Seismic Design of Substations, Power Engineering Society, Piscataway, New Jersey, 1997 (IEEE 693).

IEEE Standard 751, *Trial-Use Design Guide for Wood Transmission Structures*, Power Engineering Society, Piscataway, New Jersey, 1991. (IEEE 751)

Long, L.W., Analysis of Seismic Effects on Transmission Structures, IEEE Paper T 73 326-6, April 1973. (IEEE T 73 326-6).

Lum, W. B., Nielson, N. N., Koyanagi, R., and Chui, A. N. L., Damage Survey of the Kasiki, Hawaii Earthquake of November 16, 1993, Earthquake Spectra, November 1984. (Lum 1984)

Lyver T.D., Mueller W.H., Kempner L, Jr., Response Modification Factor, R_w, for Transmission

Towers, Research Report, Portland State University, Portland, Oregon, 1996. (Lyver 1996)Towers, Research Report, Portland State University, Portland, Oregon, 1996. (Lyver 1996)

National Center for Earthquake Engineering Research (NCEER):
The Hanshin-Awaji Earthquake of January 17, 1995 Performance of Lifelines, National Center for Earthquake Engineering Research, Technical Report NCEER-95-0015, State University of New York at Buffalo, November 3, 1995. (NCEER-95-0015)

Rural Electrical Administration (REA):
Bulletin 1724E-200, *Design Manual for High Voltage Transmission Lines*, 1992 (REA 1724).

Bulletin 65-1, Design Guide for Rural Substations, 1978 (REA 65-1).

Bulletin 160-2, *Mechanical Design Manual for Overhead Distribution Lines*, 1982. (REA 160)

Telecommunications Industry Association (TIA):
TIA/EIA 222F *Structural Standards for Steel Antenna Towers and Antenna Supporting Structures*, 1996. (TIA 222)

C14A.2 ELECTRICAL TRAMSMISSION, SUBSTATION, AND DISTRIBUTION STRUCTURES: The design of electrical transmission, substation wire support, and distribution structures is typically controlled by high wind, ice-wind combinations, and unbalance longitudinal wire loads (Agrawal 1976, ASCE 74, ASCE 72, ASCE 10, ASCE 2000, ASCE 97, REA 65-1, NESC). Distribution structures typically support equipment with low mass and seismic loads do not control their design (REA 160, REA 1724, IEEE 751, NESC). Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional electrical transmission, substation, and distribution wire support structure loading (Steinhardt 1981). These structures may be used in special situations were seismic loads should be considered in their design. The special situations for transmission and substation wire support structures may include site specific low wind velocity and ice load, and no designed unbalance longitudinal wire load. For distribution structures, the number of supported transformers may result in significant seismic load. Seismic lateral loads and design criteria for substation structures should satisfy the requirements of IEEE 693, 1997.

Earthquake-related damage to electrical transmission, substation wire support, and distribution structures typically is caused by large displacements of the foundations due to landslides, ground failure, and liquefaction (FEMA, 1990). These situations have resulted in structural failure or damaged structural members without complete loss of structure function.

The fundamental frequency of these structure types typically ranges from 0.5 to 6 Hz. Single pole type structures have fundamental mode frequencies in the 0.5 to 1.5 Hz range. H-frame structures have fundamental mode frequencies in the 1 to 3 Hz ranges, with the lower frequencies in the direction normal to the plane of the structure and the higher frequencies in plane. Four legged lattice structures have fundamental mode frequencies in the range of 2 to 6 Hz. Lattice tangent structures typically have lower frequencies with the higher frequencies being representative of angle and dead end structures. These frequency ranges can be used to determine if earthquake loading should be a design consideration. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs. The default viscous damping value to be used in such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

A minimum importance factor (I) of 1.0 should be used to provide the necessary seismic resistance. An I of 1.0 is required to minimize the loss of function after an earthquake event even though these systems are normally redundant.

The R values shown in Table C14A.3.2 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. An analysis of lattice (truss) type transmission towers dictated R values in the range of 3 to 8 (Lyver 1996). The value of 3 for truss systems shown in Table C14A.3.2 represents the lower bound value of R. In general, the remaining R values shown reflect the earthquake performance of these structural systems and engineering judgment. Other values may be appropriate if determined using sound engineering data.

The C_d and W values shown in Table C14A.3.2 for these types of *structures* are presented for information only and to be consistent with parameters presented for other facilities covered by the *Provisions*. The C_d value is a factor used to estimate the peak inelastic deflection (d_{inel}) during a seismic event when the elastic displacements (d_{el}) from a static analysis using seismic loads are known ($d_{inel} = d_{el}C_d$). The W values represent a component force factor to provide increased reliability in strength for a critical component (component force times W). The magnitude of this factor is currently specified (when used) by the industry design standards and recommended practices specified in Sec. 14.3.

Traditionally, wire supported mass and dynamic effects have not been included in the evaluation of structural response (Long 1973). Some studies have suggested that for long spans the seismic contribution of the wires should not be neglected (Li 1991, Li 1994). Reasons for neglecting the supported wires are the order of magnitude difference between the wire system natural frequency and that of the supporting structures and the method of connection between these two systems. The spatial distribution of the structural system (varying wire spans, tower location and geometry, and seismic ground motion) also helps mitigate the effects of dynamic coupling. The satisfactory performance of these structures during earthquakes does not justify the additional loading as a result of the wire dynamics. Engineering judgment should be used to determined the inclusion or the significance of the wire mass.

C14A.3: TELECOMMUNICATION TOWERS: This section was placed in the Appendix to Chapter 14 for the following reasons:

1. To provide a starting point for continued development.
2. To stimulate comment and input for development of this section to the end that it will be incorporated in the *Provisions* in the future.
3. It was determined by TS13 and the Provisions Update Committee that it would be premature to incorporate this section into the *Provisions* for the 2000 edition.
4. Accepted industry standards are in the process of incorporating seismic design methodology reflecting

the *Provisions*.

It is not the intent of the Provisions Update Committee to discourage incorporation of this section into a building code or to minimize the importance of this section. Placing this section in the appendix indicates only that this section requires further development.

The design of telecommunication towers is typically controlled by extreme wind, ice and wind combinations, and restrictive deflection (serviceability) limits (TIA 222; CSA 1994). Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional telecommunication loading (Lum, 1983). As a minimum, this requirement should be to determine the significance of seismic loads in the design of the tower. Seismic lateral loads in combination with long-term ice loads should be considered. Recommendations for combined load effects can be found in ASCE 7.

A general industry survey indicated that the seismic performance of these structures to earthquake loading has been acceptable. Reported earthquake damage has been limited to failure of building mounted towers and shifting of mounted antennas resulting in misalignment of the signal path (FEMA,202, Lum, 1983; NCEER, 1995; Steinhardt, 1981).

The fundamental frequency of these structural types typically ranges from 0.5 to 10 Hz. If it is determined that earthquake loads are significant then a more detailed evaluation of the structure's vibration frequencies and mode shapes should be performed. This can be accomplished using available commercial finite element computer programs. The default viscous damping value to be used with such an analysis should be 2 percent. A higher damping value can be used if determined using sound engineering data.

Recent studies (Galvez, 1995) have suggested that a linear lateral force distribution ($k = 1$) is not an accurate representation for self-supporting telecommunication towers. The lateral force distribution being studied accounts for the mass participation of the lowest three flexural modes of vibration of the tower. Until further studies have been completed and a final recommendation is available it is recommended that a linear distribution be used with the *Provisions* when a refined lateral force distribution is required.

The R values shown in Table C14A.3.2 reflect the inelastic reserve strength of the structural systems during an earthquake event. The values presented for these types of structures were determined based on a review of published values established for building structures and nonbuilding structures. Other values may be appropriate if determined using sound engineering data.

The C_d and Ω values shown in Table C14A.3.2 for these types of structures are presented for information only and to be consistent with parameters presented for other facilities covered by the *Provisions*. The C_d value is a factor used to estimate the peak inelastic deflection (d_{inel}) during a seismic event when the elastic displacements (d_{el}) from a static analysis using seismic loads are known ($d_{inel} = d_{el}C_d$). The Ω values represent a component force factor to be used to provide increased reliability in strength for a critical component (component force times Ω). The magnitude of this factor is currently specified (when used) by the industry design standards and recommended practices specified in Sec.14.3.

Guyed towers taller than 66 m should be evaluated using modal analysis procedures. Modeling of a guyed tower must allow for geometric nonlinearities and potential interactions between the mast and the guy wires (Amiri, 1996). The significant earthquake effect will be due to the dynamic interaction between the mast and the guy wires. The analysis of guyed towers can be accomplished using available commercial finite element computer programs.

Reference AS 3995 has an informative appendix that provides guidance on when earthquake design of guyed and self-supporting telecommunication towers may be appropriate. The following information is obtained from this document.

1. Steel lattice and guyed towers are less sensitive to earthquake loads than most other structure types.
2. Self-supporting lattice towers up to 100 m high and having insignificant mass concentrations less than 25 percent of their total mass need not be designed for earthquakes.
3. Self-supporting lattice towers of insignificant mass and over 100 m high or lesser height with significant mass concentrations may experience base shears and base overturning moments approaching those caused by ultimate wind loads.
4. Self-supporting lattice towers and guyed steel masts that are in earthquake design zones should be designed considering the vertical component of ground motion. For very tall guyed towers, some vertical ground motion differentials between the mast base and guy anchorage points may be an important design consideration depending on local seismicity.

C14A.4 BURIED STRUCTURES:

This section was placed in the Appendix to Chapter 14 for the following reasons:

1. To provide a starting point for continued development.
2. To stimulate comment and input for development of this section to the end that it will be incorporated in the *Provisions* in the future.
3. It was determined by TS13 and the Provisions Update Committee that it would be premature to incorporate this section into the *Provisions* for the 2000 edition.
4. Accepted industry standards are in the process of incorporating seismic design methodology reflecting the *Provisions*.

It is not the intent of the Provisions Update Committee to discourage incorporation of this section into a building code or to minimize the importance of this section. Placing this section in the appendix indicates only that this section requires further development.

Seismic forces on buried structures may include forces due to: soil displacement, seismic lateral earth pressure, buoyant forces related to liquefaction, permanent ground displacements from slope instability, lateral spread movement, or fault movement, dynamic ground displacement from dynamic strains from wave propagation. Identification of appropriate seismic loading conditions is dependent upon subsurface soil conditions and the configuration of the buried structure. Conditions related to permanent ground movement can often be avoided by careful site selection for isolated buried structures such as tanks and vaults. Relocation is often impractical for long buried structures such as tunnels and pipelines.

Wave propagation strains are a significant seismic force condition to buried structures if local site conditions can support the propagation of large amplitude seismic waves (e.g., deep surface soil deposits with low shear wave velocities). Wave propagation strains tend to be most pronounced at the junctions of dissimilar buried structures (e.g., a pipeline connecting with a building) or at the interfaces of different geologic materials (e.g., a pipeline passing from rock to soft soil).

Loading conditions related to liquefaction require detailed subsurface information that can assess the potential for liquefaction and, for long buried structures, the length of structure exposed to liquefaction effects. In addition, the assessment of liquefaction requires specifying an earthquake magnitude that is consistent with the definition of ground shaking. It is recommended that one refer to Chapter 7 *Commentary* for additional guidance in determining liquefaction potential and seismic magnitude. Providing detailed structural design procedures in this area is beyond the scope of this document.

Loading conditions related to lateral spread movement and slope instability can be defined in terms of lateral soil pressures or prescribed ground displacements. In both cases, sufficient subsurface investigation in the vicinity of the buried structure is necessary to estimate the amount of movement, the direction of movement relative to the buried structure, and the portion of the buried structure exposed to the loading conditions. Definition of lateral spread loading conditions requires special geotechnical expertise and specific procedures in this area are beyond the scope of this document.

Defining the loading conditions for fault movement requires specific location of the fault and an estimate of the earthquake magnitude on the fault that is consistent with the ground shaking hazard in the *Provisions*. Identification of the fault location should be based on past earthquake movements, trenching studies, information from boring logs, or other accepted fault identification techniques. Defining fault movement conditions requires special seismological expertise. Additional guidance can be found in the Chapter 7 *Commentary*.

It may not be practically feasible to design a buried structure to resist the effects of permanent ground deformation. Alternative approaches in such cases may include relocation to avoid the condition, ground improvements to reduce the loads, or implementing special procedures or design features to minimize the impact of damage (e.g., remote controlled or automatic isolation valves, that provide the ability to rapidly bypass damage, post-earthquake procedures to expedite repair). The goal of providing procedures or design features as an alternative to designing for the seismic loadings is to change the hazard and function classification of the buried structure such that it is not classified as *Seismic Use Group* II or III.

It is recommended that one refer to Chaper 7 *Commentary* for additional guidance in determining liquefaction potential, and determining seismic magnitude.

Buried *structures* are subgrade *structures* such as tanks, tunnels, and pipes. Buried *structures* that are designated as *Seismic Use Group* II or III, or are of such a size or length to warrant special seismic design as determined by the registered design professional shall be identified in the geotechnical report.

Buried *structures* shall be designed to resist minimum seismic lateral forces determined from a substantiated analysis using approved procedures. Flexible couplings shall be provided for buried *structures* requiring special seismic considerations where changes in the support system, configuration, or soil condition occur.

Commentary Appendix A

DEVELOPMENT OF MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAPS 1 THROUGH 24

BACKGROUND

The maps used in the *Provisions* through 1994 provided the A_a (effective peak acceleration coefficient) and A_v (effective peak velocity-related acceleration coefficient) values to use for design. The BSSC had always recognized that the maps and coefficients would change with time as the profession gained more knowledge about earthquakes and their resulting ground motions and as society gained greater insight into the process of establishing acceptable risk.

By 1997, significant additional earthquake data had been obtained that made the A_a and A_v maps, then about 20 years old, seriously out of date. For the 1997 *Provisions*, a joint effort involving the BSSC, the Federal Emergency Management Agency (FEMA), and the U.S. Geological Survey (USGS) was conducted to develop both new maps for use in design and new design procedures reflecting the significant advances made in the past 20 years. The BSSC's role in this joint effort was to develop new ground motion maps for use in design and design procedures based on new USGS seismic hazard maps.

The BSSC appointed a 15-member Seismic Design Procedure Group (SDPG) to develop the seismic ground motion maps and design procedures. The SDPG membership was composed of representatives of different segments of the design community as well as two earth science members designated by the USGS, and the membership was representative of the different geographical regions of the country. Also, the BSSC, with input from FEMA and USGS, appointed a five-member Management Committee (MC) to guide the efforts of the SDPG. The MC was geographically balanced insofar as practicable and was composed of two seismic hazard definition experts and three engineering design experts, including the chairman of the 1997 *Provisions* Update Committee (PUC). The SDPG and the MC worked closely with the USGS to define the BSSC mapping needs and to understand how the USGS seismic hazard maps should be used to develop the BSSC seismic ground motion maps and design procedures.

For a brief overview of how the USGS developed its hazard maps, see Appendix B to this *Commentary* volume. A detailed description of the development of the maps is contained in the USGS Open-File Report 96-532, *National Seismic-Hazard Maps: Documentation, June 1996*, by Frankel, et al. (1996). The USGS hazard maps also can be viewed and printed from a USGS Internet site at http://gldage.cr.usgs.gov/eq/html/finmain.shtml.

The goals of the SDPG were as follows:

1. To replace the existing effective peak acceleration and velocity-related acceleration design maps with new ground motion spectral response maps based on new USGS seismic hazard maps.

2. To develop the new ground motion spectral response maps within the existing framework of the *Provisions* with emphasis on uniform margin against the collapse of structures.

3. To develop design procedures for use with the new ground motion spectral response maps.

PURPOSE OF THE *PROVISIONS*

The purpose of the *Provisions* is to present criteria for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. To this end, the *Provisions* provides the minimum criteria considered prudent for structures subjected to earthquakes at any location in the United States and its territories. The *Provisions* generally considers property damage as it relates to occupant safety for ordinary structures. For high occupancy and essential structures, damage limitation criteria are more strict in order to better provide for the safety of occupants and the continued functioning of the structure. Some structural and nonstructural damage can be expected as a result of the "design ground motions" because the *Provisions* allows inelastic energy dissipation by utilizing the deformability of the structural system. For ground motions in excess of the design levels, the intent is that there be a low likelihood of collapse. These goals of the *Provisions* were the guiding principles for developing the design maps.

POLICY DECISIONS FOR SEISMIC GROUND MOTION MAPS

The new maps (cited in both the 1997 and 2000 *Provisions*) reflect the following policy decisions that depart from past practice and the 1994 *Provisions*:

1. The maps define the maximum considered earthquake ground motion for use in design procedures,

2. The use of the maps for design provide an approximately uniform margin against collapse for ground motions in excess of the design levels in all areas.

3. The maps are based on both probabilistic and deterministic seismic hazard maps, and

4. The maps are response spectra ordinate maps and reflect the differences in the short-period range of the response spectra for the areas of the United States and its territories with different ground motion attenuation characteristics and different recurrence times.

These policy decisions reflected new information from both the seismic hazard and seismic engineering communities that is discussed below.

In the 1994 *Provisions*, the design ground motions were based on an estimated 90 percent probability of not being exceeded in 50 years (about a 500 year mean recurrence interval) (ATC 3-06 1978). The 1994 *Provisions* also recognized that larger ground motions are possible and that the larger motions, although their probability of occurrence during a structure's life is very small, nevertheless can occur at any time. The 1994 *Provisions* also defined a maximum capable earthquake as "the maximum level of earthquake ground shaking that may ever be expected at the building site within the known geologic framework." It was additionally specified that in certain map areas ($\geq A_a = 0.3$), the maximum capable earthquake was associated with a motion that has a

90 percent probability of not being exceeded in 100 years (about a 1000 year mean recurrence interval). In addition to the maximum capable earthquake definition, sample ground motion maps were prepared with 90 percent probabilities of not being exceeded in 250 years (about a 2500 year mean recurrence interval).

Given the wide range in return periods for maximum magnitude earthquakes throughout the United States and its territories (100 years in parts of California to 100,000 years or more in several other locations), current efforts have focused on defining the maximum considered earthquake ground motions for use in design (not the same as the maximum capable earthquake defined in the 1994 *Provisions*). The maximum considered earthquake ground motions are determined in a somewhat different manner depending on the seismicity of an individual region; however, they are uniformly defined as the maximum level of earthquake ground shaking that is considered as reasonable to design structures to resist. Focusing on ground motion versus earthquake size facilitates the development of a design approach that provides an approximately uniform margin against collapse throughout the United States.

As noted above, the 1994 *Provisions* generally used the notation of 90 percent probability of not being exceeded in a certain exposure time period (50, 100, or 250 years), which can then be used to calculate a given mean recurrence interval (500, 1000, or 2500 years). For the purpose of the new maps and design procedure introduced in the 1997 *Provisions*, the single exposure time period of 50 years has been commonly used as a reference period over which to consider loads on structures (after 50 years of use, structures may require evaluation to determine future use and rehabilitation needs). With this in mind, different levels of probability or return period are expressed as percent probability of exceedance in 50 years. Specifically, 10 percent probability of exceedance in 50 years is a mean recurrence interval of about 500 years, 5 percent probability of exceedance in 50 years is a mean recurrence interval of about 1000 years, and 2 percent probability of exceedance in 50 years is a mean recurrence interval of about 2500 years. The above notation is used throughout the *Provisions*.

Review of modern probabilistic seismic hazard results, including the maps prepared by the USGS to support the effort resulting in the 1997 *Provisions*, indicates that the rate of change of ground motion versus probability is not constant throughout the United States. For example, the ground motion difference between the 10 percent probability of exceedance and 2 percent probability of exceedance in 50 years in coastal California is typically smaller than the difference between the two probabilities in less active seismic areas such as the eastern or central United States. Because of these differences, questions were raised concerning whether definition of the ground motion based on a constant probability for the entire United States would result in similar levels of seismic safety for all structures. Figure A1 plots the 0.2 second spectral acceleration normalized at 2 percent probability of exceedance in 50 years versus the annual frequency of exceedance. Figure A1 shows that in coastal California, the ratio between the 0.2 second spectral acceleration for the 2 and the 10 percent probabilities of exceedance in 50 years is about 1.5 whereas, in other parts of the United States, the ratio varies from 2.0 to 5.0.

In answering the questions, it was recognized that seismic safety is the result of a number of steps in addition to defining the design earthquake ground motions, including the critical items generally defined as proper site selection, structural design criteria, analysis and procedures, detailed design requirements, and construction.

The conservatism in the actual design of the structure is often referred to as the "seismic margin." It is the seismic margin that provides confidence that significant loss of life will not be caused by actual ground motions equal to the design levels. Alternatively, the seismic margin provides a level of protection against larger, less probable earthquakes although at a lower level of confidence.

The collective opinion of the SDPG was that the seismic margin contained in the *Provisions* provides, as a minimum, a margin of about 1.5 times the design earthquake ground motions. In other words, if a structure experiences a level of ground motion 1.5 times the design level, the

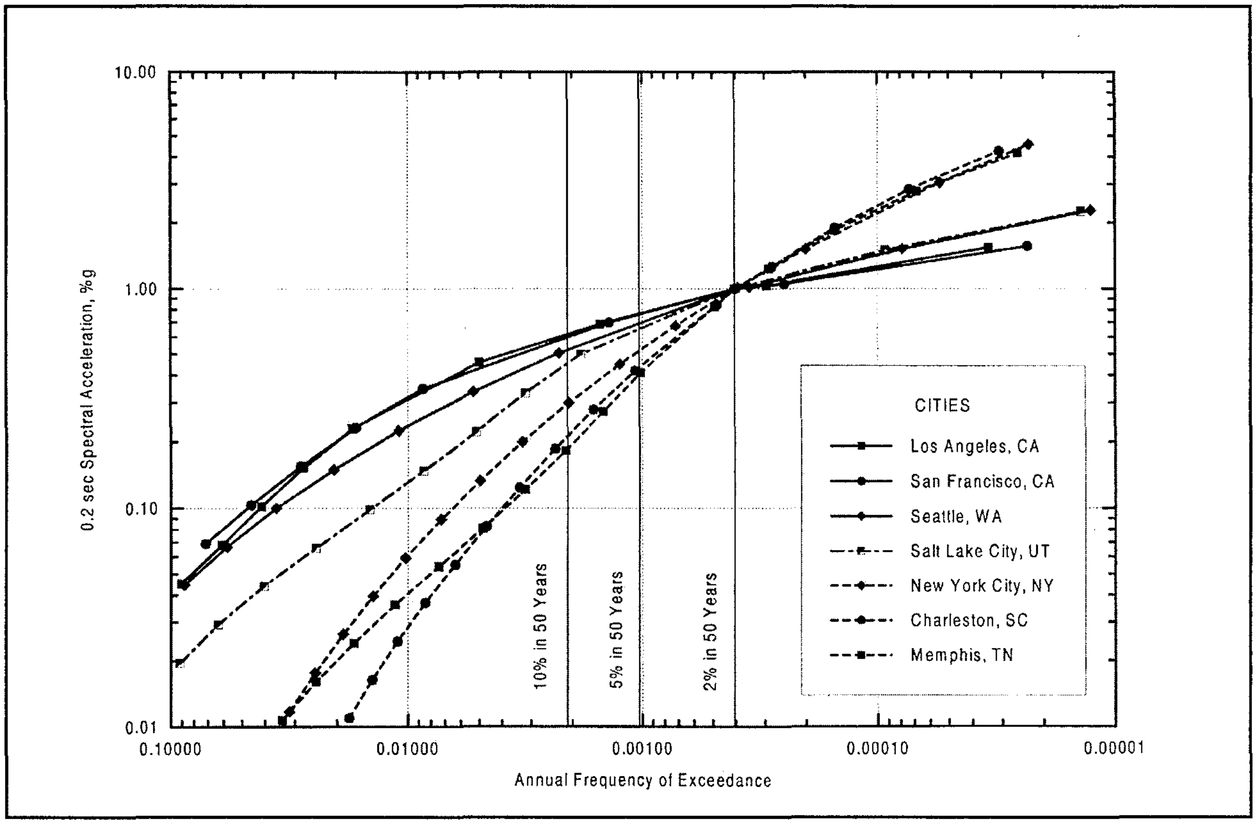


FIGURE A1 Relative hazard at selected sites for 0.2 sec spectral response acceleration. The hazard curves are normalized at 2 percent probability of exceedance in 50 years.

structure should have a low likelihood of collapse. The SDPG recognizes that quantification of this margin is dependent on the type of structure, detailing requirements, etc., but the 1.5 factor is a conservative judgment appropriate for structures designed in accordance with the *Provisions*. This seismic margin estimate is supported by Kennedy et al. (1994), Cornell (1994), and Ellingwood (1994) who evaluated structural design margins and reached similar conclusions.

The USGS seismic hazard maps indicate that in most locations in the United States the 2 percent probability of exceedance in 50 years ground motion values are more than 1.5 times the 10 percent probability of exceedance in 50 years ground motion values. This means that if the 10 percent probability of exceedance in 50 years map was used as the design map and the 2 percent probability of exceedance in 50 years ground motions were to occur, there would be low confidence (particularly in the central and eastern United States) that structures would not collapse due to these larger ground motions. Such a conclusion for most of the United States was not acceptable to the SDPG. The only location where the above results seemed to be acceptable was coastal California (2 percent probability of exceedance in 50 years map is about 1.5 times the 10 percent probability of exceedance in 50 years map) where structures have experienced levels of ground shaking equal to and above the design value.

The USGS probabilistic seismic hazard maps for coastal California also indicate the 10 percent probability of exceedance in 50 years seismic hazard map is significantly different from (in most cases larger) the design ground motion values contained in the 1994 *Provisions*. Given the generally successful experience with structures that complied with the recent editions of the *Uniform Building Code* whose design map contained many similarities to the 1994 *Provisions* design map, the SDPG was reluctant to suggest large changes without first understanding the basis for the changes. This stimulated a detailed review of the probabilistic maps for coastal California. This review identified a unique issue for coastal California in that the recurrence interval of the estimated maximum magnitude earthquake is less than the recurrence interval represented on the probabilistic map, in this case the 10 percent probability of exceedance in 50 years map (i.e., recurrence interval for maximum magnitude earthquake is 100 to 200 years versus 500 years.)

Given the above, one choice was to accept the change and use the 10 percent probability of exceedance in 50 years probabilistic map to define the design ground motion for coastal California and, using this, determine the appropriate probability for design ground motion for the rest of the United States that would result in the same level of seismic safety. This would have resulted in the design earthquake being defined at 2 percent probability of exceedance in 50 years and the need for development of a 0.5 to 1.0 percent probability of exceedance in 50 years map to show the potential for larger ground motions outside of coastal California. Two major problems were identified. The first is that requiring such a radical change in design ground motion in coastal California seems to contradict the general conclusion that the seismic design codes and process are providing an adequate level of life safety. The second is that completing probabilistic estimates of ground motion for lower probabilities (approaching those used for critical facilities such as nuclear power plants) is associated with large uncertainties and can be quite controversial.

An alternative choice was to build on the observation that the maximum earthquake for many seismic faults in coastal California is fairly well known and associated with probabilities larger than a 10 percent probability of exceedance in 50 years (500 year mean recurrence interval). Given this, a decision was made to develop a procedure that would use the best estimate of ground motion from maximum magnitude earthquakes on seismic faults with high probabilities of occurrence (short return periods). For the purposes of the *Provisions,* these earthquakes are defined as "deterministic earthquakes." Following this approach and recognizing the inherent seismic margin contained in the *Provisions,* it was determined that the level of seismic safety achieved in coastal California would be approximately equivalent to that associated with a 2 to 5 percent probability of exceedance in 50 years for areas outside of coastal California. In other words, the use of the deterministic earthquakes to establish the maximum considered earthquake ground motions for use in design in coastal California results in a level of protection close to that implied in the 1994 *Provisions* and consistent with maximum magnitude earthquakes expected for those seismic sources. Additionally, this approach results in less drastic changes to ground motion values for coastal California than the alternative approach of using probabilistic based maps.

One could ask why any changes are necessary for coastal California given the positive experience from recent earthquakes. While it is true that the current seismic design practices have produced positive results, the current design ground motions in the 1994 *Provisions* are less than those expected from maximum magnitude earthquakes on known seismic sources. The 1994 *Provisions* reportedly considered maximum magnitude earthquakes but did not directly link them to the design ground motions (Applied Technology Council, 1978). If there is high confidence in the definition of the fault and magnitude of the earthquake and the maximum earthquake occurs frequently, then the design should be linked to at least the best estimate ground motion for such an earthquake. Indeed, it is the actual earthquake experience in coastal California that is providing increased confidence in the seismic margins contained in the *Provisions.*

The above approach also is responsive to comments that the use of 10 percent probability of exceedance in 50 years is not sufficiently conservative in the central and eastern United States where the earthquakes are expected to occur infrequently. Based on the above discussion and the inherent seismic margin contained in the *Provisions*, the SDPG selected 2 percent probability of exceedance in 50 years as the maximum considered earthquake ground motion for use in design where the use of the deterministic earthquake approach discussed above is not used.

The maximum considered earthquake ground motion maps are based on two response spectral values (a short-period and a long-period value) instead of the A_a and A_v coefficients. The decision to use response spectral values is based on earthquake data obtained during the past 20 years showing that site-specific spectral values are more appropriate for design input than the A_a and A_v coefficients used with standardized spectral shapes. The spectral shapes vary in different areas of the country and for different site conditions. This is particularly the case for the short-period portion of the response spectra. Based on the differences in the ground motion attenuation characteristics between the central and eastern and western United States, the USGS used different ground motion attenuation functions for these areas in developing the seismic hazard maps. The ground motion attenuation functions in the eastern United States result in higher short-period spectral accelerations at lower periods for a given earthquake magnitude than the western United States attenuation functions, particularly compared to the high seismicity region of coastal California. The short-period response spectral values were reviewed in order to determine the most appropriate value to use for the maximum considered earthquake ground motion maps. Based on this review, the short-period spectral response value of 0.2 second was selected to represent the short-period range of the response spectra for the eastern United States. In the western United States the most appropriate short-period response spectral value was determined to be 0.3 second, but a comparison of the 0.2 and 0.3 second values indicated that the differences in the response spectral values were insignificant. Based on this and for convenience of preparing the maximum considered earthquake ground motion maps, the short-period response spectral value of 0.2 second was selected to represent the short-period range of the response spectra for all of the United States. The long-period response spectral value selected for use is 1.0 second for all of the United States. Based on the ground motion attenuation functions and the USGS seismic hazard maps, a $1/T$ (T = natural period) relationship was selected to define the response spectra from the short period value to the long-period value. Using the spectral values from the ground motion maps will allow the different spectral shapes to be incorporated into design.

DEVELOPMENT OF THE MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAPS FOR USE IN DESIGN

The concept for developing maximum considered earthquake ground motions for use in design involved two distinct steps:

1. The various USGS probabilistic seismic hazard maps were combined with deterministic hazard maps by a set of rules (logic) to create the maximum considered earthquake ground motion maps that can be used to define response spectra for use in design and
2. Design procedures were developed that transform the response spectra into design values (e.g., design base shear).

The response spectra defined from the first step represent general "site-dependent" spectra similar to those that would be obtained by a geotechnical study and used for dynamic analysis except their shapes are less refined (i.e., shape defined for only a limited number of response periods). The response spectra do not represent the same hazard level across the country but do represent actual ground motion consistent with providing approximately uniform protection against the collapse of structures. The response spectra represent the maximum considered earthquake ground motions for use in design for Site Class B (rock with a shear wave velocity of 760 meters/second).

The maximum considered earthquake ground motion maps for use in design are based on a defined set of rules for combining the USGS seismic hazard maps to reflect the differences in the ability to define the fault sources and seismicity characteristics across the regions of the country as discussed in the policy decisions. Accommodating regional differences allows the maximum considered earthquake maps to represent ground motions for use in design that provide reasonably consistent margins of preventing the collapse of structures. Based on this, three regions have been defined:

1. Regions of negligible seismicity with very low probability of collapse of the structure,
2. Regions of low and moderate to high seismicity, and
3. Regions of high seismicity near known fault sources with short return periods.

Regions of Negligible Seismicity With Very Low Probability of Collapse of the Structure

The regions of negligible seismicity with very low probability of collapse have been defined by:

1. Determining areas where the seismic hazard is controlled by earthquakes with M_b (body wave magnitude) magnitudes less than or equal to 5.5 and
2. Examining the recorded ground motions associated with Modified Mercalli Intensity V.

The basis for the first premise is that in this region, there are a number of examples of earthquakes with $M_b \cong 5.5$ which caused only localized damage to structures not designed for earthquakes. The basis for the second premise is that Modified Mecalli Intensity V ground motions typically do not cause structural damage. By definition, Modified Mercalli Intensity V ground shaking is felt by most people, displaces or upsets small objects, etc., but typically causes no, or only minor, structural damage in buildings of any type. Modified Mercalli Intensity VI ground shaking is felt by everyone, small objects fall off shelves, etc., and minor or moderate structural damage occurs to weak plaster and masonry construction. Life-threatening damage or collapse of *structures* would not be expected for either Modified Mercalli Intensities V or VI ground shaking. Based on an evaluation of 1994 Northridge earthquake data, regions of different Modified Mercalli Intensity (Dewey, 1995) were correlated with maps of smooth response spectra developed from instrumental recordings (Sommerville, 1995). The Northridge earthquake provided a sufficient number of instrumental recordings and associated spectra to permit correlating Modified Mercalli Intensity with response spectra. The results of the correlation determined the average response spectrum for each Modified Mercalli Intensity region. For Modified Mercalli Intensity V, the average response spectrum of that region had a spectral response acceleration of slightly greater than 0.25g at 0.3 seconds and a spectral response acceleration of slightly greater than 0.10g at 1.0 seconds. On the basis of these values and the minor nature of damage associated with Modified Mercalli Intensity V, 0.25g (short-period acceleration) and 0.10g (acceleration at a period of 1 second, taken proportional to 1/T) is deemed to be a conservative estimate of the spectrum below which life-threatening damage would not be expected to occur even to the most vulnerable of types of structures. Therefore, this region is defined as areas having maximum considered earthquake ground motions with a 2 percent probability of exceedance in 50 years equal to or less than 0.25g (short period) and 0.10g (long period). The seismic hazard in these areas is generally the result of $M_b \cong 5.5$ earthquakes. In these areas, a minimum lateral force design of 1 percent of the dead load of the structure shall be used in addition to the detailing requirements for the Seismic Design Category A structures.

In these areas it is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion. The ground motion computed for such areas is determined more by the rarity of the event with respect to the chosen level of probability than by the level of motion that would occur if a small but close earthquake actually did occur. However, it is desirable to provide some protection, both against earthquakes as well as many other types of unanticipated loadings. The requirements for Seismic Design Category A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible, but rare earthquake. The result of design to Seismic Design Category A is that fewer buildings would collapse in the vicinity of such an earthquake.

The integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be identified. Then it must be designed for a lateral force equal to a 1% acceleration on the mass. Lastly, the minimum connection forces specified for Seismic Design Category A must be satisfied.

The 1 percent value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local building codes will normally control the lateral force design when compared to the minimum structural integrity force on the structure. However, many low-rise heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1 percent acceleration. Also, minimum connection forces may exceed structural forces due to wind in additional structures.

The regions of negligible seismicity will vary depending on the Site Class on which structures are located. The *Provisions* seismic ground motion maps (Maps 1 through 19) are for Site Class B conditions and the region of negligible seismicity for Site Class B is defined where the maximum considered earthquake ground motion short-period values are ≤0.25g and the long-period values are ≤ 0.10g. The regions of negligible seismicity for the other Site Classes are defined by using the appropriate site coefficients to determine the maximum considered earthquake ground motion for the Site Class and then determining if the short-period values are ≤ 0.25g and the long-period values are ≤ 0.10g. If so, then the site of the structure is located in the region of negligible seismicity for that Site Class.

Regions of Low and Moderate to High Seismicity

In regions of low and moderate to high seismicity, the earthquake sources generally are not well defined and the maximum magnitude estimates have relatively long return periods. Based on this, probabilistic hazard maps are considered to be the best means to represent the uncertainties and to define the response spectra for these regions. The maximum considered earthquake ground motion for these regions is defined as the ground motion with a 2 percent probability of exceedance in 50 years. The basis for this decision is explained in the policy discussion.

Consideration was given to establishing a separate region of low seismicity and defining a minimum level of ground motion (i.e., deterministic minimum ground motions). This was considered because in the transition between the regions of negligible seismicity to the regions of low seismicity, the ground motions are relatively small and may not be very meaningful for use in seismic design. The minimum level was also considered because the uncertainty in the ground motion levels in the regions of low seismicity is larger than in the regions of moderate to high seismicity. This larger uncertainty may warrant consideration of using higher ground motions (or some minimum level of ground motion) than provided by the maximum considered earthquake ground motions shown on the maps.

The studies discussed above for the regions of negligible seismicity by Dewey (1995) and Sommerville (1995), plus other unpublished studies (to date), were evaluated as a means of determining minimum levels of ground motion for used in design. These studies correlated the Modified Mercalli Intensity data with the recorded ground motions and associated damage. The studies included damage information for a variety of structures which had no specific seismic design and determined the levels of ground motion associated with each Modified Mercalli Intensity. These studies indicate that ground motion levels of about 0.50g short-period spectral response and 0.20g long-period spectral response are representative of Modified Mercalli Intensity VII damage.

Modified Mercalli Intensity VII ground shaking results in negligible damage in buildings of good design and construction, slight to moderate damage in well-built ordinary buildings, considerable damage in poorly-built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), etc. In other words, Modified Mercalli VII ground shaking is about the level of ground motion where significant structural damage may occur and result in life safety concerns for occupants. This tends to suggest that designing structures for ground motion levels below 0.50g short-period spectral response and 0.20g long-period spectral response may not be meaningful.

One interpretation of this information suggests that the ground motion levels for defining the regions of negligible seismicity could be increased. This interpretation would result in much larger regions that require no specific seismic design compared to the 1994 *Provisions*.

Another interpretation of the information suggests establishing a minimum level of ground Motion (at about the Modified Mercalli VII shaking) for regions of low seismicity, in order to transition from the regions of negligible seismicity to the region of moderate to high seismicity. Implementation of a minimum level of ground motion, such as 0.50g for the short-period spectral response and 0.20g for the long-period spectral response, would result in increases (large percentages) in ground motions used for design compared to the 1994 *Provisions*.

Based on the significant changes in past practices resulting from implementing either of the above interpretations, the SDPG decided that additional studies are needed to support these changes. Results of such studies should be considered for future editions of the *Provisions*.

Regions of High Seismicity Near Known Fault Sources With Short Return Periods

In regions of high seismicity near known fault sources with short return periods, deterministic hazard maps are used to define the response spectra maps as discussed above. The maximum considered earthquake ground motions for use in design are determined from the USGS deterministic hazard maps developed using the ground motion attenuation functions based on the median estimate increased by 50 percent. Increasing the median ground motion estimates by 50 percent is deemed to provide an appropriate margin and is similar to some deterministic estimates for a large magnitude characteristic earthquake using ground motion attenuation functions with one standard deviation. Estimated standard deviations for some active fault sources have been determined to be higher than 50 percent, but this increase in the median ground motions was considered reasonable for defining the maximum considered earthquake ground motions for use in design.

Maximum Considered Earthquake Ground Motion Maps for Use in Design

Considering the rules for the three regions discussed above, the maximum considered earthquake ground motion maps for use in design were developed by combining the regions in the following manner:

1. Where the maximum considered earthquake map ground motion values (based on the 2 percent probability of exceedance in 50 years) for Site Class B adjusted for the specific site conditions are ≤ 0.25g for the short-period spectral response and ≤ 0.10g for the long period spectral response, then the site will be in the region of negligible seismicity and a minimum lateral force design of 1 percent of the dead load of the structure shall be used in addition to the detailing requirements for the Seismic Design Category A structures.
2. Where the maximum considered earthquake ground motion values (based on the 2 percent probability of exceedance in 50 years) for Site Class B adjusted for the specific site conditions are greater than 0.25g for the short-period spectral response and 0.10g for the long-period spectral response, the maximum considered earthquake ground motion values (based on the 2 percent probability of exceedance in 50 years adjusted for the specific site conditions) will be used until the values equal the present (1994 *Provisions*) ceiling design values increased by 50 percent (short period = 1.50g, long period = 0.60g). The present ceiling de-

sign values are increased by 50 percent to represent the maximum considered earthquake ground motion values. This will define the sites in regions of low and moderate to high seismicity.

3. To transition from regions of low and moderate to high seismicity to regions of high seismicity with short return periods, the maximum considered earthquake ground motion values based on 2 percent probability of exceedance in 50 years will be used until the values equal the present (1994 *Provisions*) ceiling design values increased by 50 percent (short period = 1.50g, long period = 0.60g). The present ceiling design values are increased by 50 percent to represent maximum considered earthquake ground motion values. When the 1.5 times the ceiling values are reached, then they will be used until the deterministic maximum considered earthquake map values of 1.5g (long period) and 0.60g (short period) are obtained. From there, the deterministic maximum considered earthquake ground motion map values will be used.

In some cases there are regions of high seismicity near known faults with return periods such that the probabilistic map values (2 percent probability of exceedance in 50 years) will exceed the present ceiling values of the 1994 *Provisions* increased by 50 percent and will be less than the deterministic map values. In these regions, the probabilistic map values will be used for the maximum considered earthquake ground motions.

The basis for using present ceiling design values as the transition between the two regions is because earthquake experience has shown that regularly configured, properly designed *structures* performed satisfactorily in past earthquakes. The most significant structural damage experienced in the Northridge and Kobe earthquakes was related to configuration, structural systems, inadequate connection detailing, incompatibility of deformations, and design or construction deficiencies -- not due to deficiency in strength (Structural Engineers Association of California, 1995). The earthquake designs of the structures in the United States (coastal California) which have performed satisfactorily in past earthquakes were based on the criteria in the *Uniform Building Code*. Considering the site conditions of the structures and the criteria in the *Uniform Building Code*, the ceiling design values for these structures were determined to be appropriate for use with the *Provisions* maximum considered earthquake ground motion maps for Site Class B. Based on this, the equivalent maximum considered earthquake ground motion values for the ceiling were determined to be 1.50g for the short period and 0.60g for the long period.

As indicated above there also are some regions of high seismicity near known fault sources with return periods such that the probabilistic map values (2 percent probability of exceedance in 50 years) will exceed the ceiling values of the 1994 *Provisions* increased by 50 percent and also be less than the deterministic map values. In these regions, the probabilistic map values are used for the maximum considered earthquake ground motions.

The near source area in the high seismicity regions is defined as the area where the maximum considered earthquake ground motion values are $\geq$ 0.75g on the 1.0 second map. In the near source area, *Provisions* Sec. 5.2.3 through 5.2.6 impose additional requirements for certain structures unless the structures are fairly regular, do not exceed 5 stories in height, and do not have a period of vibration over 0.5 seconds. For the fairly regular structures not exceeding 5 stories in height and not having a period of vibration over 0.5 seconds, the maximum considered earthquake ground motion values will not exceed the present ceiling design values increased by 50 percent. The basis for this is because of the earthquake experience discussed above.

These development rules for the maximum considered earthquake ground motion maps for use in design are illustrated in Figures A2 and A3. The application of these rules resulted in the maximum considered earthquake ground motion maps (Maps 1 through 24) introduced in the 1997 and used again in the 2000 *Provisions*.

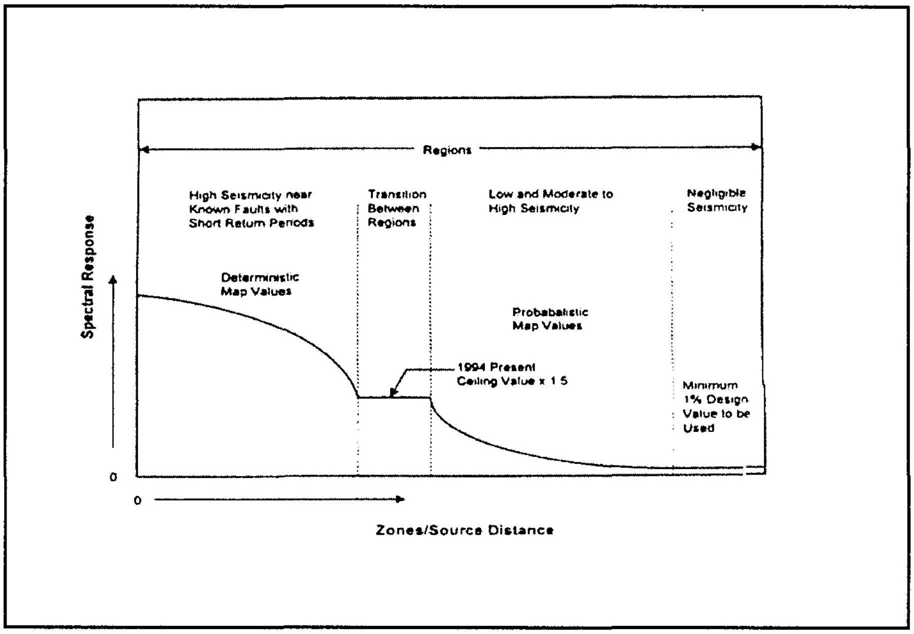


FIGURE A2 Development of the maximum considered earthquake ground motion map for spectral acceleration of T = 1.0, Site Class B.

STEP 1 -- DEFINE POTENTIAL SEISMIC SOURCES

A. ***Compile Earth Science Information*** -- Compile historic seismicity and fault characteristics including earthquake magnitudes and recurrence intervals.

B. ***Prepare Seismic Source Map*** -- Specify historic seismicity and faults used as sources.

STEP 2 -- PREPARE PROBABILISTIC AND DETERMINISTIC SPECTRAL RESPONSE MAPS

A. ***Develop Regional Attenuation Relations***

(1) Eastern U.S. (Toro, et al., 1993, and Frankel, 1996)

(2) Western U.S. (Boore et al., 1993 &1994, Campbell and Bozorgnia, 1994, and Sadigh, 1993 for PGA. Boore et al., 1993 &1994, and Sadigh, 1993 for spectral values)

(3) Deep Events (>35km) (Geomatrix et al., 1993)

(4) Cascadia Subduction Zone (Geomatrix et al., 1993, and Sadigh, 1993)

B. ***Prepare Probabilistic Spectral Response Maps (USGS Probabilistic Maps)*** -- Maps showing S_S and S_1 where S_S and S_1 are the short and 1 second period ground motion response spectral values for a 2 percent chance of exceedence in 50 years inferred for sites with average shear wave velocity of 760 m/s from the information developed in Steps 1A and 1B and the ground motion attenuation relationships in Step 2A.

C. ***Prepare Deterministic Spectral Response Maps (USGS Deterministic Maps)*** -- Maps showing S_S and S_1 for faults and maximum earthquakes developed in Steps 1A and 1B and the median ground motion attenuation relations in Step 2A increased by 50% to represent the uncertainty.

STEP 3 -- PREPARE EARTHQUAKE GROUND MOTION SPECTRAL RESPONSE MAPS FOR PROVISIONS (MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION MAP)

Region 1 -- Regions of Negligible Seismicity with Very Low Probability of Collapse of the *Structure* (No Spectral Values)

Region definition: Regions for which S_S < 0.25g and S_1 < 0.10g from Step 2B.

Design values: No spectral ground motion values required. Use a minimum lateral force level of 1 percent of the dead load for Seismic Design Category A.

Region 2 -- Regions of Low and Moderate to High Seismicity (Probabilistic Map Values)

Region definition: Regions for which 0.25g < S_S < 1.5g and 0.25g < S_1 < 0.60g from Step 2B.

Maximum considered earthquake map values: Use S_S and S_1 map values from Step 2B.

Transition Between Regions 2 and 3 - Use MCE values of S_S = 1.5g and S_1 = 0.60g

Region 3 -- Regions of High Seismicity Near Known Faults (Deterministic Values)

Region definition: Regions for which 1.5g < S_S and 0.60g < S_1 from Step 2C.

Maximum considered earthquake map values: Use S_S and S_1 map values from Step 2C.

FIGURE A3 Methodology for development of the maximum considered earthquake ground motion maps (Site Class B).

Use of the Maximum Considered Earthquake Ground Motion Maps in the Design Procedure: The 1994 *Provisions* defined the seismic base shear as a function of the outdated effective peak velocity-related acceleration A_v, and effective peak acceleration, A_a. Beginning with the 1997 *Provisions*, the base shear of the structure is defined as a function of the maximum considered earthquake ground motion maps where S_S = maximum considered earthquake spectral acceleration in the short-period range for Site Class B; S_1 = maximum considered earthquake spectral acceleration at the 1.0 second period for Site Class B; $S_{MS} = F_a S_S$, maximum considered earthquake spectral acceleration in the short-period range adjusted for Site Class effects where F_a is the site coefficient defined in *Provisions* Sec. 4.1.2; $S_{M1} = F_v S_1$, maximum considered earthquake spectral acceleration at 1.0 second period adjusted for Site Class effects where F_v is the site coefficient defined in *Provisions* Sec. 4.1.2; S_{DS} = (2/3) S_{MS}, spectral acceleration in the short-period range for the design ground motions; and S_{D1} = (2/3) S_{M1}, spectral acceleration at 1.0 second period for the design ground motions.

As noted above, the design ground motions S_{DS} and S_{D1} are defined as 2/3 times the *maximum considered earthquake* ground motions. The 2/3 factor is based on the estimated seismic margins in the design process of the *Provisions* as previously discussed (i.e., the design level of ground motion is 1/1.5 or 2/3 times the maximum considered earthquake ground motion).

Based on the above defined ground motions, the base shear is:

$$V = C_s W$$

where $C_s = \dfrac{S_{DS}}{R/I}$ and S_{DS} = the design spectral response acceleration in the short period range as determined from Sec. 4.1.2, R = the response modification factor from Table 5.2.2, and I = the occupancy importance factor determined in accordance with Sec. 1.4.

The value of C_s need not exceed $C_s = \dfrac{S_{D1}}{T(R/I)}$ but shall not be taken less than

$C_s = 0.1 S_{D1}$ or, for buildings and structures in Seismic Design Categories E and F,

$C_s = \dfrac{0.5 S_1}{R/I}$ where I and R are as defined above and S_{D1} = the design spectral response acceleration at a period of 1.0 second as determined from Sec. 4.1.2, T = the fundamental period of the structure (sec) determined in Sec. 5.4.2, and S_1 = the mapped maximum considered earthquake spectral response acceleration determined in accordance with Sec. 4.1.

Where a design response spectrum is required by these *Provisions* and site-specific procedures are not used, the design response spectrum curve shall be developed as indicated in Figure A4 and as follows:

1. For periods less than or equal to T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 4.1.2.6-1:

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS} \tag{4.1.2.6-1}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

3. For periods greater than T_S, the design spectral response acceleration, S_a, shall be taken as given by Eq. 4.1.2.6-3:

$$S_a = \frac{S_{D1}}{T} \tag{4.1.2.6-3}$$

where:

S_{DS} = the design spectral response acceleration at short periods;

S_{D1} = the design spectral response acceleration at 1 second period;

T = the fundamental period of the *structure* (sec);

T_0 = $0.2S_{D1}/S_{DS}$; and

T_S = S_{D1}/S_{DS}.

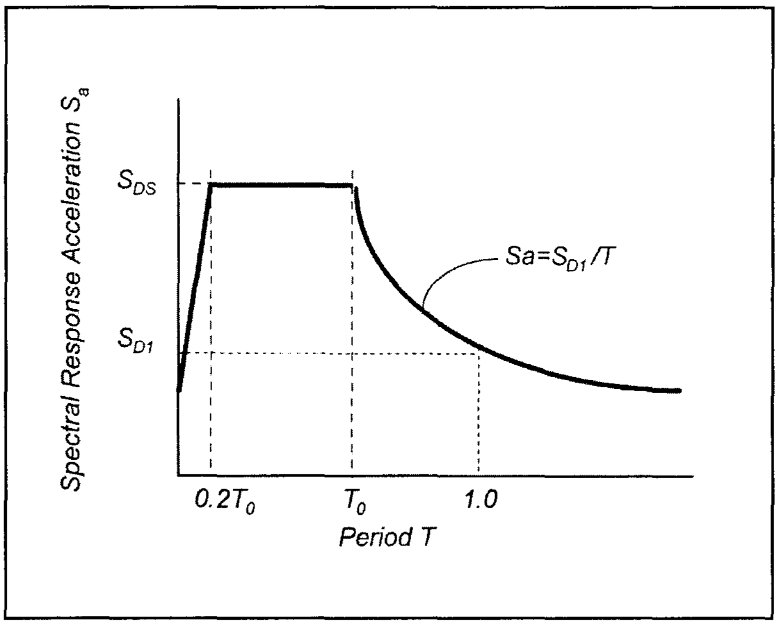


FIGURE A4 Design response spectrum.

Site-specific procedures for determining ground motions and response spectra are discussed in Sec. 4.1.3 of the *Provisions*.

REFERENCES

Applied Technology Council. 1978. *Tentative Provisions for the Development of Seismic Regulations for Building*, Report ATC-3-06. Redwood City, California: Applied Technology Council.

Structural Engineers Association of California. 1995. *Vision 2000, Performance Based Seismic Engineering of Buildings*. Prepared by the Structural Engineers Association of California, Vision 2000 Committee, for the California Office of Emergency Services.

Kennedy, R. P. and S. A. Short. 1994. *Basis for Seismic Provisions of DOE-STD-1020,* UCRL-CR-111478. Prepared for Lawrence Livermore National Laboratory. Livermore: University of California.

Ellingwood, B. R. 1994. "Probability-based Codified Design for Earthquakes." *Engineering Structures* 17(7).

Cornell, C. A. 1994. *Risk-Based Structural Design, Proceedings of Symposium on Risk Analysis,* Ann Arbor: University of Michigan.

Dewey, J. W., B. G. Reagor, L. Dengler, and K. Moley. 1995. *Intensity Distribution and Isoseismal Maps for the Northridge, California, Earthquake of January 17, 1994*, U. S. Geological Survey Open-File Report 95-92.

Sommerville, P. 1995. "Smooth Site Response Spectra Contours Developed for ATC-33," Appendix B of *Guidelines for Seismic Rehabilitation of Existing Buildings, Northridge Earthquake Case-Study Report*, ATC-33-1. Redwood City, California: Applied Technology Council.

Frankel, et al. 1996. *National Seismic-Hazard Maps: Documentation, June 1996*, U.S. Geological Survey Open-File Report 96-532.

Commentary Appendix B

DEVELOPMENT OF THE USGS SEISMIC MAPS

INTRODUCTION

The 1997 *NEHRP Recommended Provisions* used new design procedures based on the use of spectral response acceleration rather than the traditional peak ground acceleration and/or peak ground velocity and these procedures are used again in the 2000 *Provisions*. The use of spectral ordinates and their relationship to building codes has been described by Leyendecker et al (1995). The spectral response accelerations used in the new design approach are obtained from combining probabilistic maps (Frankel, et al, 1996) prepared by the U.S. Geological Survey (USGS) with deterministic maps using procedures developed by the Building Seismic Safety Council's Seismic Design Procedures Group (SDPG). The SDPG recommendations are based on using the 1996 USGS probabilistic hazard maps with additional modifications based on review by the SDPG and the application of engineering judgement. This appendix summarizes the development of the USGS maps and describes how the 1997 and 2000 *Provisions* design maps were prepared from them using SDPG recommendations. The SDPG effort has sometimes been referred to as Project '97.

DEVELOPMENT OF PROBABILISTIC MAPS FOR THE UNITED STATES

New seismic hazard maps for the conterminous United States were completed by the USGS in June 1996 and placed on the Internet World Wide Web (http://geohazards.cr.usgs. gov/eq/). The color maps can be viewed on the Web and/or downloaded to the user's computer for printing. Paper copies of the maps are also available (Frankel et al, 1997a, 1997b).

New seismic hazard maps for Alaska were completed by the USGS in January 1998 and placed on the USGS web site (http://geohazards.cr.usgs. gov/eq/). Both documentation and printing of the maps are in progress (U. S. Geological Survey, 1998a, 1998b).

New probabilistic maps are in preparation for Hawaii using the methodology similar to that used for the rest of the United States. and described below. These maps will be to be completed in early 1998. Probabilistic maps for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, St. Croix, Guam, and Tutuila needed for the 1997 *Provisions* are not expected during the current cycle of USGS map revisions (development of design maps for these areas is described below).

This appendix provides a brief description of the USGS seismic hazard maps, the geologic/seismologic inputs to these maps, and the ground-motion relations used for the maps. It is based on the USGS map documentation for the central and eastern United States (CEUS) and the western United States prepared by Frankel et al (1996). The complete reference document, also available on the USGS Web site, should be reviewed for detailed technical information.

The hazard maps depict probabilistic ground acceleration and spectral response acceleration with 10 percent, 5 percent, and 2 percent probabilities of exceedance (PE) in 50 years. These maps correspond to return times of approximately 500, 1000, and 2500 years, respectively.* All spectral response values shown in the maps correspond to 5 percent of critical damping. The maps are based on the assumption that earthquake occurrence is Poissonian, so that the probability of occurrence is time-independent. The methodologies used for the maps were presented, discussed, and substantially modified during 6 regional workshops for the conterminous United States convened by the USGS from June 1994-June 1995. A seventh workshop for Alaska was held in September 1996.

The methodology for the maps (Frankel et al., 1996) includes three primary features:

1. The use of smoothed historical seismicity is one component of the hazard calculation. This is used in lieu of source zones used in previous USGS maps. The analytical procedure is described in Frankel (1995).
2. Another important feature is the use of alternative models of seismic hazard in a logic tree formalism. For the central and eastern United States (CEUS), different models based on different reference magnitudes are combined to form the hazard maps. In addition, large background zones based on broad geologic criteria are used as alternative source models for the CEUS and the western United States (WUS). These background zones are meant to quantify hazard in areas with little historic seismicity, but with the potential to produce major earthquakes. The background zones were developed from extensive discussions at the regional workshops.
3. For the WUS, a big advance in the new maps is the use of geologic slip rates to determine fault recurrence times. Slip rates from about 500 faults or fault segments were used in preparing the probabilistic maps.

The hazard maps do not consider the uncertainty in seismicity or fault parameters. Preferred values of maximum magnitudes and slip rates were used instead. The next stage of this effort is the quantification of uncertainties in hazard curves for selected sites. These data will be included on the Internet as they become available.

The USGS hazard maps are not meant to be used for Mexico, areas north of 49 degrees north latitude, and offshore the Atlantic and Gulf of Mexico coasts of the United States.

CEUS and WUS Attenuation Boundary

Attenuation of ground motion differs between the CEUS and the WUS. The boundary between regions was located along the eastern edge of the Basin and Range province (Figure B1). The

* Previous USGS maps (e.g. Algermissen, et al, 1990 and Leyendecker, et al, 1995) and earlier editions of the *Provisions* expressed probability as a 10 percent probability of exceedance in a specified exposure time. Beginning with the 1996 maps, probability is being expressed as a specified probability of exceedance in a 50 year time period. Thus, 5 percent in 50 years and 2 percent in 50 years used now correspond closely to 10 percent in 100 years and 10 percent in 250 years, respectively, that was used previously. This same information may be conveyed as annual frequency. In this approach 10 percent probability of exceedance (PE) in 50 years corresponds to an annual frequency of exceedance of 0.0021; 5 percent PE in 100 years corresponds to 0.00103; and 2 percent PE in 50 years corresponds to 0.000404.

previous USGS maps (e.g., Algermissen et al., 1990) used an attenuation boundary further to the east along the Rocky Mountain front.

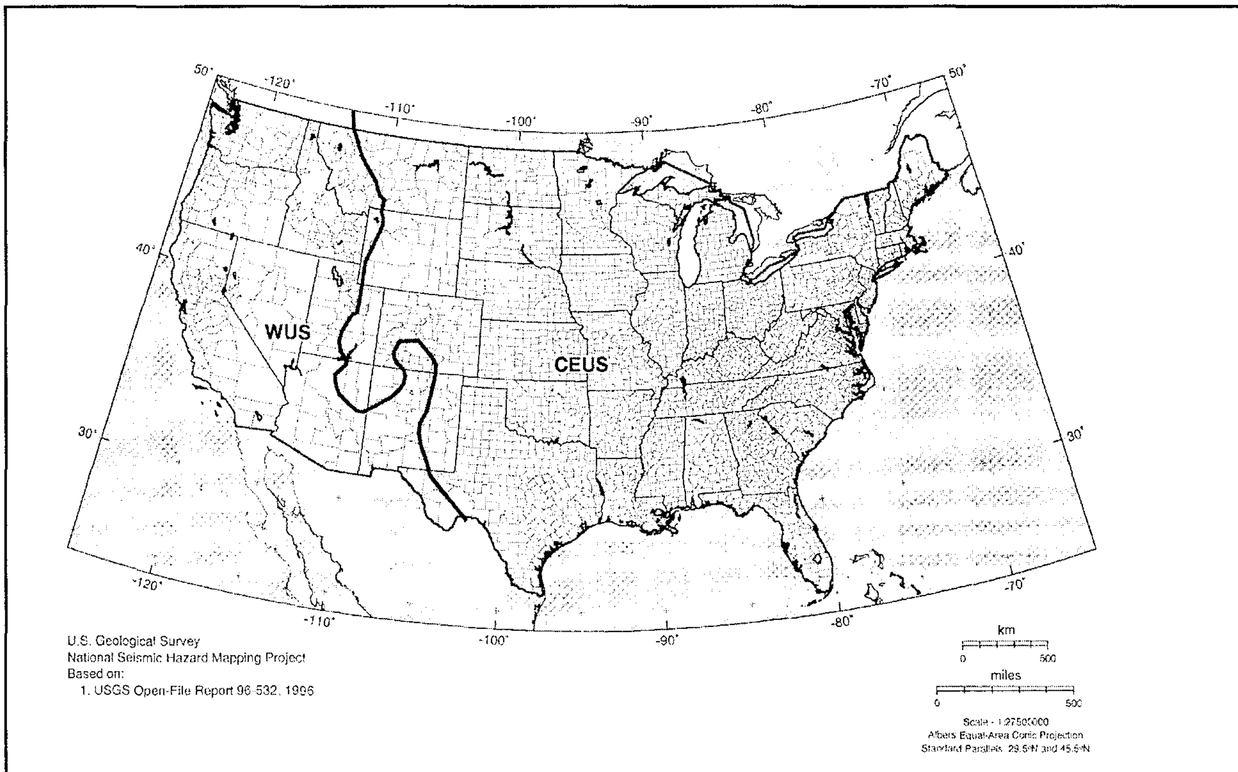


FIGURE B1 Attenuation boundary for eastern and western attenuation function.

Separate hazard calculations were done for the two regions using different attenuation relations. Earthquakes west of the boundary used the WUS attenuation relations and earthquakes east of the boundary used CEUS attenuation relations. WUS attenuation relations were used for WUS earthquakes, even for sites located east of the attenuation boundary. Similarly CEUS attenuations were used for CEUS earthquakes, even for sites located west of the attenuation boundary. It would have been computationally difficult to consider how much of the path was contained in the attenuation province. Also, since the attenuation relation is dependent on the stress drop, basing the relation that was used on the location of the earthquake rather than the receiver is reasonable.

Hazard Curves

The probabilistic maps were constructed from mean hazard curves, that is the mean probabilities of exceedance as a function of ground motion or spectral response. Hazard curves were obtained for each site on a calculation grid.

A grid (or site) spacing of 0.1 degrees in latitude and longitude was used for the WUS and 0.2 degrees for the CEUS. This resulted in hazard calculations at about 65,000 sites for the WUS runs and 35,000 sites for the CEUS runs. The CEUS hazard curves were interpolated to yield a set of hazard curves on a 0.1 degree grid. A grid of hazard curves with 0.1 degree spacing was thereby obtained for the entire conterminous United States. A special grid spacing of 0.05 degrees was also done for California, Nevada, and western Utah because of the density of faults warranted increased density of data. These data were used for maps of this region.

Figure B2 is a sample of mean hazard curves used in making the 1996 maps. The curves include cities from various regions in the United States. It should be noted that in some areas the curves are very sensitive to the latitude and longitude selected. A probabilistic map is a contour plot of the ground motion or spectral values obtained by taking a "slice" through all 150,000 hazard curves at a particular probability value. The gridded data obtained from the hazard curves that was used to make each probabilistic map is located at the USGS Web site. Figure B2 also shows the general difference in slope of the hazard curves of the CEUS versus the WUS. This difference has been noted in other studies.

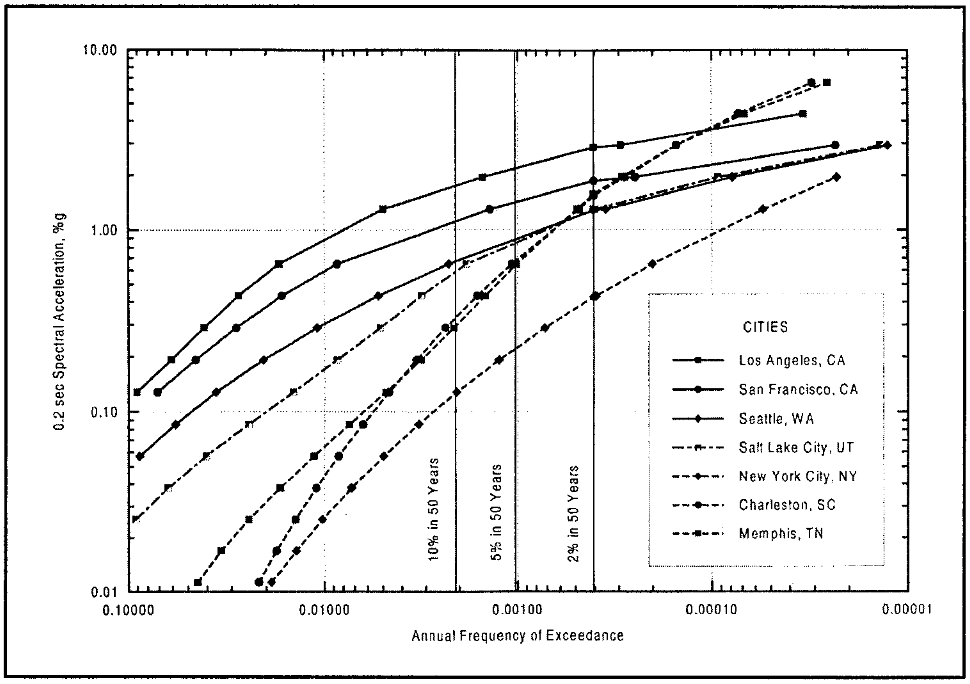


FIGURE B2 Hazard curves for selected cities.

CENTRAL AND EASTERN UNITED STATES

The basic procedure for constructing the CEUS portion of the hazard maps is diagramed in Figure B3. Four models of hazard are shown on the left side of the figure. Model 1 is based on m_b 3.0 and larger earthquakes since 1924. Model 2 is derived from m_b 4.0 and larger earthquakes since 1860. Model 3 is produced from m_b 5.0 and larger events since 1700. In constructing the hazard maps, model 1 was assigned a weight twice that of models 2 and 3.

The procedure described by Frankel (1995) is used to construct the hazard maps directly from the historic seismicity (models 1 - 3). The number of events greater than the minimum magnitude are counted on a grid with spacing of 0.1 degrees in latitude and longitude. The logarithm of this number represents the maximum likelihood a-value for each grid cell. Note that the maximum likelihood method counts a m_b 5 event the same as a m_b 3 event in the determination of a-value. Then the gridded a-values are smoothed using a Gaussian function. A Gaussian with a correlation distance of 50 km was used for model 1 and 75 km for models 2 and 3. The 50 km distance was chosen because it is similar in width to many of the trends in historic seismicity in the CEUS. In addition, it is comparable to the error in location of m_b 3 events in the period of 1924-1975, before the advent of local seismic networks. A larger correlation distance was used for models 2 and 3 since they include earthquakes further back in time with poorer estimates of locations.

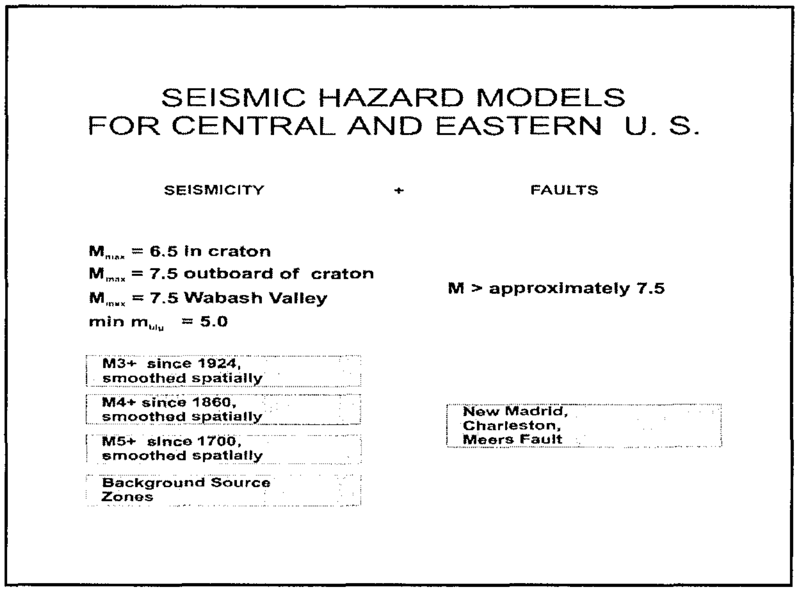


FIGURE B3 Seismic hazard models for the central and eastern United States. Smoothed seismicity models are shown on the left and fault models are shown on the right.

Model 4 consists of large background source zones. This alternative is meant to quantify hazard in areas with little historical seismicity but with the potential to generate damaging earthquakes. These background zones are detailed in a later section of this text. The sum of the weights of models 1-4 is one. For a weighting scheme that is uniform in space, this ensures that the total seismicity rate in the combined model equals the historic seismicity rate. A spatially-varying weighting scheme which slightly exceeds the historic seismicity rate was used in the final map for reasons which are described later.

A regional b-value of 0.95 was used for models 1-4 in all of the CEUS except Charlevoix, Quebec. This b-value was determined from a catalog for events east of 105 degrees W. For the Charlevoix region a b-value of 0.76 was used based on the work of John Adams, Stephen Halchuck and Dieter Weichert of the Geologic Survey of Canada (see Adams et al., 1996).

Figure B4 shows a map of the CEUS M_{max} values used for models 1-4 (bold M refers to moment magnitude). These M_{max} zones correspond to the background zones used in model 4. Most of the CEUS is divided into a cratonic region and a region of extended crust. An M_{max} of 6.5 was used for the cratonic area. A M_{max} of 7.5 was used for the Wabash Valley zone in keeping with magnitudes derived from paleoliquefaction evidence (Obermeier et al., 1992). An M_{max} of 7.5 was used in the zone of extended crust outboard of the craton. An M_{max} of 6.5 was used for the Rocky Mountain zone and the Colorado Plateau, consistent with the magnitude of the largest historic events in these regions. An M_{max} of 7.2 was used for the gridded seismicity within the Charleston areal source zone. A minimum m_b of 5.0 was used in all the hazard calculations for the CEUS.

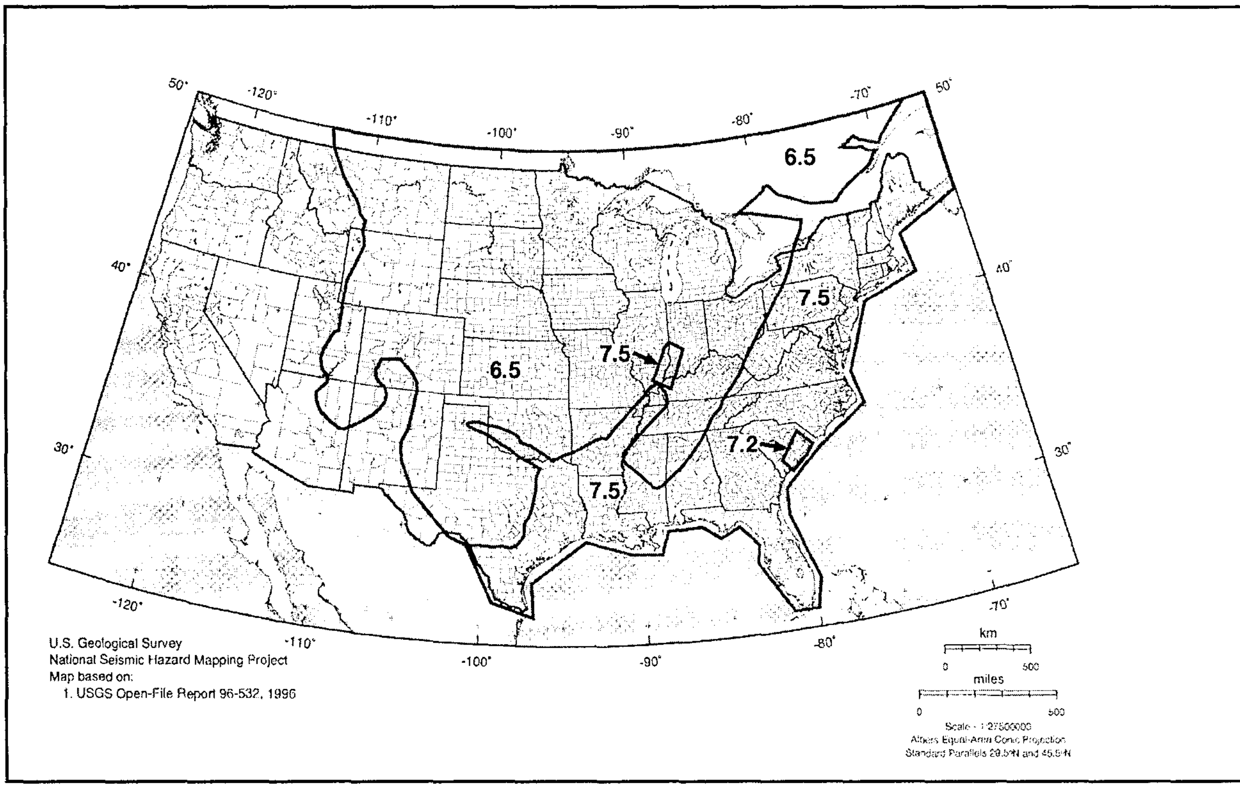


FIGURE B4 central and eastern U.S. maximum magnitude zones.

Model 5 (Figure B3, right) consists of the contribution from large earthquakes (M>7.0) in four specific areas of the CEUS: New Madrid, Charleston, South Carolina, the Meers fault in southwest Oklahoma, and the Cheraw Fault in eastern Colorado. This model has a weight of 1. The treatment of these special areas is described in B.3.1. There are three other areas in the CEUS that are called special zones: eastern Tennessee, Wabash Valley, and Charlevoix. These are described in B.3.1.

Special Zones

New Madrid: To calculate the hazard from large events in the New Madrid area, three parallel faults in an S-shaped pattern encompassing the area of highest historic seismicity were considered. These were not meant to be actual faults; they are simply a way of expressing the uncertainty in the source locations of large earthquakes such as the 1811-12 sequence. A characteristic rupture model with a characteristic moment magnitude M of 8.0, similar to the estimated magnitudes of the largest events in 1811-12 (Johnston, 1996a,b) was assumed. A recurrence time of 1000 years for such an event was used as an average value, considering the uncertainty in the magnitudes of pre-historic events.

An areal source zone was used for New Madrid for models 1-3, rather than spatially-smoothed historic seismicity. This zone accounts for the hazard from New Madrid events with moment magnitudes less than 7.5.

Charleston, South Carolina: An areal source zone was used to quantify the hazard from large earthquakes. The extent of the areal source zone was constrained by the areal distribution of paleoliquefaction locations, although the source zone does not encompass all the paleoliquefaction sites. A characteristic rupture model of moment magnitude 7.3 earthquakes, based on the estimated magnitude of the 1886 event (Johnston, 1996b) was assumed. For the M7.3 events a recurrence time of 650 years was used, based on dates of paleoliquefaction events (Amick and Gelinas, 1991; Obermeier et al., 1990, Johnston and Schweig, written comm., 1996).

Meers Fault: The Meers fault in southwestern Oklahoma was explicitly included. The segment of the fault which has produced a Holocene scarp as described in Crone and Luza (1990) was used. A characteristic moment magnitude of 7.0 and a recurrence time of 4000 years was used based on their work.

Cheraw Fault: This eastern Colorado fault with Holocene faulting based on a study by Crone et al. (1996) was included. The recurrence rate of this fault was obtained from a slip rate of 0.5 mm/yr. A maximum magnitude of 7.1 was found from the fault length using the relations of Wells and Coppersmith (1994).

Eastern Tennessee Seismic Zone: The eastern Tennessee seismic zone is a linear trend of seismicity that is most obvious for smaller events with magnitudes around 2 (see Powell et al., 1994). The magnitude 3 and larger earthquakes tend to cluster in one part of this linear trend, so that hazard maps are based just on smoothed m_b3.

Wabash Valley: Recent work has identified several paleoearthquakes in the areas of southern Indiana and Illinois based on widespread paleoliquefaction features (Obermeier et al., 1992). An areal zone was used with a higher M_{max} of 7.5 to account for such large events. The sum of the gridded a-values in this zone calculated from model 1 produce a recurrence time of 2600 years for events with m_b 6.5. The recurrence rate of M6.5 and greater events is estimated to be about 4,000 years from the paleoliquefaction dates (P. Munson and S. Obermeier, pers. comm., 1995), so it is not necessary to add additional large events to augment models 1-3. The Wabash Valley M_{max} zone in the maps is based on the Wabash Valley fault zone.

Charlevoix, Quebec: As mentioned above, a 40 km by 70 km region surrounding this seismicity cluster was assigned a b-value of 0.76, based on the work of Adams, Halchuck and Weichert. This b-value was used in models 1-3.

Background Source Zones (Model 4)

The background source zones (see Figure B5) are intended to quantify seismic hazard in areas that have not had significant historic seismicity, but could very well produce sizeable earthquakes in the future. They consist of a cratonic zone, an extended margin zone, a Rocky Mountain zone, and a Colorado Plateau zone. The Rocky Mountain zone was not discussed at any workshop, but is clearly defined by the Rocky Mountain front on the east and the areas of extensional tectonics to the west, north and south. As stated above, the dividing line between the cratonic and extended margin zone was drawn by Rus Wheeler based on the westward and northern edge of rifting during the opening of the Iapetan ocean. One justification for having craton and extended crust zones is the work done by Johnston (1994). They compiled a global survey of earthquakes in cratonic and extended crust and found a higher seismicity rate (normalized by area) for the extended areas.

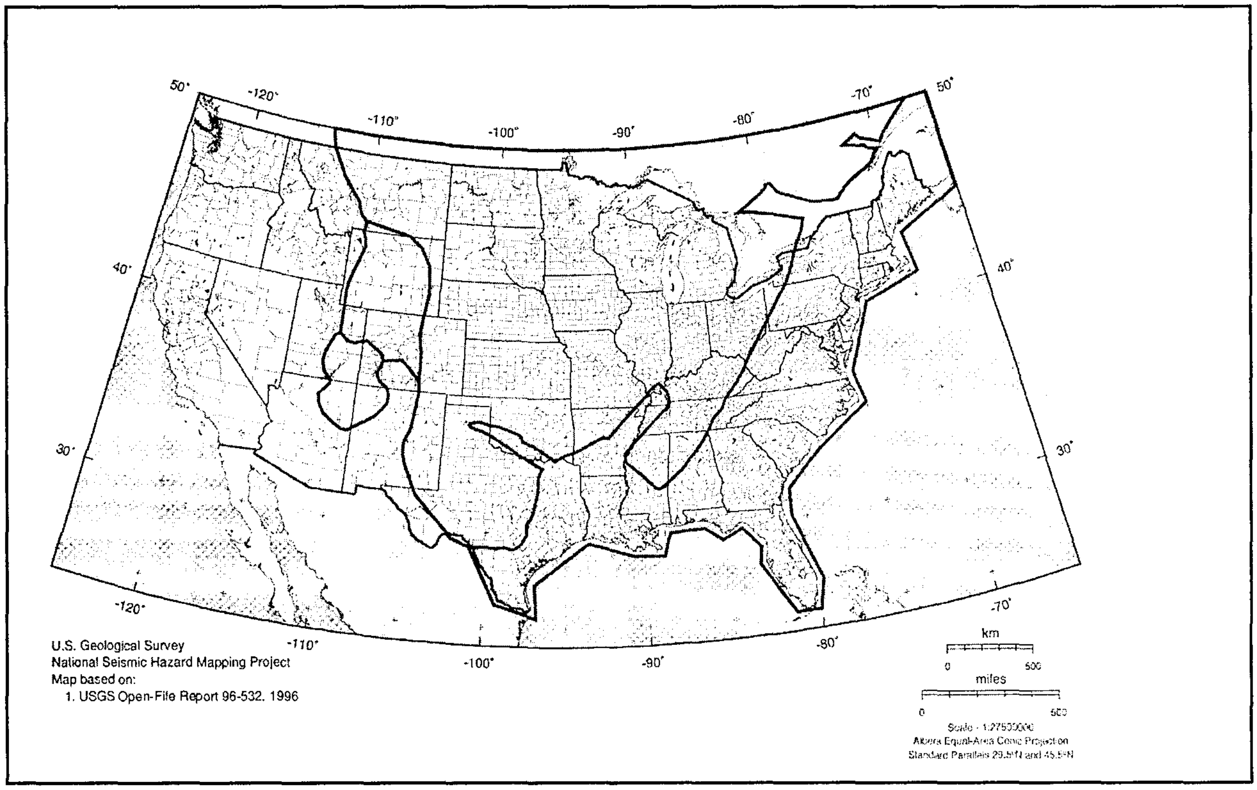


FIGURE B5 Central and eastern U.S. background zones.

For each background zone, a-values were determined by counting the number of m_b3 and larger events within the zone since 1924 and adjusting the rate to equal that since 1976. A b-value of 0.95 was used for all the background zones, based on the b-value found for the entire CEUS.

Adaptive Weighting for CEUS

The inclusion of background zones lowers the probabilistic ground motions in areas of relatively high historic seismicity while raising the hazard to only low levels in areas with no historic seismicity. The June 1996 versions of the maps include the background zones using a weighting scheme that can vary locally depending on the level of historic seismicity in that cell of the a-value grid. Spatially-varying weighting was suggested by Allin Cornell in the external review of the interim maps. The "adaptive weighting" procedure avoids lowering the hazard in higher seismicity areas to raise the hazard in low seismicity areas. This was implemented by looping through the a-value grid and checking to see if the a-value for each cell from the historic seismicity was greater than the a-value from the background zone. For the CEUS the a-value from the historic seismicity was derived by weighting the rates from models 1, 2, and 3 by 0.5, 0.25, 0.25 respectively. If this weighted sum was greater than the rate from the appropriate background zone, then the rate for that cell was determined by weighting the rates from models 1-3 by 0.5, .25, .25 (i.e., historic seismicity only, no background zone). If the weighted sum from the historic seismicity was less than the rate of the background zone, then a weighting of 0.4, 0.2, 0.2, 0.2 for models 1-4, respectively (including the background zone as model 4). This procedure does not make the rate for any cell lower than it would be from the historic seismicity (models 1-3). It also incorporates the background zones in areas of low historic seismicity. The total seismicity rate in the resulting a-value grid is only 10 percent larger than the observed rate of m_b3's since 1976. This is not a major difference. Of course, this procedure produces substantially higher ground motions (in terms of percentage increase) in the seismically quiet areas as compared to no background zone. These values are still quite low in an absolute sense.

CEUS Catalogs and B-Value Calculation

The primary catalog used for the CEUS for longitudes east of 105 degrees is Seeber and Armbruster (1991), which is a refinement of the EPRI (1986) catalog. This was supplemented with the PDE catalog from 1985-1995. In addition, PDE, DNAG, Stover and Coffman (1993), Stover, Reagor, and Algermissen (1984) catalogs were searched to find events not included in Seeber and Armbruster (1991). Mueller et al. (1996) describes the treatment of catalogs, adjustment of rates to correct for incompleteness, the removal of aftershocks, and the assignment of magnitudes.

Attenuation Relations for CEUS

The reference site condition used for the maps is specified to be the boundary between NEHRP classes B and C (Martin and Dobry, 1994), meaning it has an average shear-wave velocity of 760 m/sec in the top 30m. This corresponds to a typical "firm-rock" site for the western United States (see WUS attenuation section below), although many rock sites in the CEUS probably have much higher velocities. The motivation for using this reference site is that it corresponds to the average of sites classified as "rock" sites in WUS attenuation relations. In addition, it was considered less problematic to use this site condition for the CEUS than to use a soil condition. Most previously-published attenuation relations for the CEUS are based on a hard-rock site condition. It is less of a problem to convert these to a firm-rock condition than to convert them to a soil condition, since there would be less concern over possible non-linearity for the firm-rock site compared to the soil site.

Two equally-weighted, attenuation relations were used for the CEUS. Both sets of relations were derived by stochastic simulations and random vibration theory. First the Toro et al. (1993) attenuation for hard-rock was used. The attenuation relations were multiplied by frequency-dependent factors developed by USGS to convert them from hard-rock to firm-rock sites. The factors used 1.52 for PGA, 1.76 for 0.2 sec spectral response, 1.72 for 0.3 sec spectral response and 1.34 for 1.0 sec spectral response. These factors were applied independently of magnitude and distance.

The second set of relations was derived by USGS (Frankel et al., 1996) for firm-rock sites. These relations were based on a Brune source model with a stress drop of 150 bars. The simulations contained frequency-dependent amplification factors derived from a hypothesized shear-wave velocity profile of a CEUS firm-rock site. A series of tables of ground motions and response spectral values as a function of moment magnitude and distance was produced instead of an equation.

For CEUS hazard calculations for models 1-4, a source depth of 5.0 km was assumed when using the USGS ground motion tables. Since a minimum hypocentral distance of 10 km is used in the

USGS tables, the probabilistic ground motions are insensitive to the choice of source depth. In the hazard program, when hypocentral distances are less than 10 km the distance is set to 10 km when using the tables. For the Toro et al. (1993) relations, the fictitious depths that they specify for each period are used, so that the choice of source depth used in the USGS tables was not applied.

For both sets of ground motion relations, values of 0.75, 0.75, 0.75 and 0.80 were used for the natural logarithms of the standard deviation of PGA, 0.2 sec, 0.3 sec and 1.0 sec spectral responses, respectively. These values are similar to the aleatory standard deviations reported to the Senior Seismic Hazard Analysis Committee (1996).

A cap in the median ground motions was placed on the ground motions within the hazard code. USGS was concerned that the median ground motions of both the Toro et al. and the new USGS tables became very large (>2.5 g PGA) for distances of about 10 km for the M 8.0 events for New Madrid. Accordingly the median PGA's was capped at 1.5 g. The median 0.3 and 0.2 sec values were capped at 3.75 g which was derived by multiplying the PGA cap by 2.5 (the WUS conversion factor). This only affected the PGA values for the 2 percent PE in 50 year maps for the area directly above the three fictitious faults for the New Madrid region. It does not change any of the values at Memphis. The capping did not significantly alter the 0.3 and 0.2 sec values in this area. The PGA and spectral response values did not change in the Charleston region from this capping. Note that the capping was for the median values only. As the variability (sigma) of the ground motions was maintained in the hazard code, values larger than the median were allowed. USGS felt that the capping recognizes that values derived from point source simulations are not as reliable for M8.0 earthquakes at close-in distances (< 20 km).

Additional Notes for CEUS

One of the major outcomes of the new maps for the CEUS is that the ground motions are about a factor of 2-3 times lower, on average, than the PGA values in Algermissen et al. (1990) and the spectral values in Algermissen et al. (1991) and Leyendecker et al. (1995). The primary cause of this difference is the magnitudes assigned to pre-instrumental earthquakes in the catalog. Magnitudes of historic events used by Algermissen et al were based on I_{max} (maximum observed intensity), using magnitude-I_{max} relations derived from WUS earthquakes. This overestimates the magnitudes of these events and, in turn, overestimates the rates of M4.9 and larger events. The magnitudes of historic events used in the new maps were primarily derived by Seeber and Armbruster (1991) from either felt area or I_{max} using relations derived from CEUS earthquakes (Sibol et al., 1987). Thus, rates of M4.9 and larger events are much lower in the new catalog, compared to those used for the previous USGS maps.

It is useful to compare the new maps to the source zones used in the EPRI (1986) study. For the areas to the north and west of New Madrid, most of the six EPRI teams had three source zones in common: 1) the Nemaha Ridge in Kansas and Nebraska, 2) the Colorado-Great Lakes lineament extending from Colorado to the western end of Lake Superior, and 3) a small fault zone in northern Illinois, west of Chicago. Each of these source zones are apparent as higher hazard areas in the our maps. The Nemaha Ridge is outlined in the maps because of magnitude 4 and 5 events occurring in the vicinity. Portions of the Colorado-Great Lakes lineament show higher hazard in the map, particularly the portion in South Dakota and western Minnesota. The portion of the lineament in eastern Minnesota has been historically inactive, so is not apparent on the maps. The area in western Minnesota shows some hazard because of the occurrence of a few magnitude 4 events since 1860. A recent paper by Chandler (1995), argues that the locations and focal mechanisms of these earthquakes are not compatible with them being on the lineament, which is expressed as the Morris Fault in this region. The area in northern Illinois has relatively high hazard in the maps because of M4-5 events that have occurred there.

Frankel (1995) also found good agreement in the mean PE's and hazard curves derived from models 1-3 and 4 and those produced by the EPRI (1986) study, when the same PGA attenuation relations were used.

WESTERN UNITED STATES

The maps for the WUS include a cooperative effort with the California Division of Mines and Geology. This was made possible, in part, because CDMG was doing a probabilistic map at the same time the USGS maps were prepared. There was considerable cooperation in this effort. For example, the fault data base used in the USGS maps was obtained from CDMG. Similarly USGS software was made available to CDMG. The result is that maps produced by both agencies are the same.

The procedure for mapping hazard in the WUS is shown in Figure B6. On the left side, hazards are considered from earthquakes with magnitudes less than or equal to moment magnitude 7.0. For most of the WUS, two alternative models are used: 1) smoothed historical seismicity (weight of 0.67) and 2) large background zones (weight 0.33) based on broad geologic criteria and workshop input. Model 1 used a 0.1 degree source grid to count number of events. The determination of a-value was changed somewhat from the CEUS, to incorporate different completeness times for different magnitude ranges. The a-value for each grid cell was calculated from the maximum likelihood method of Weichert (1980), based on events with magnitudes of 4.0 and larger. The ranges used were M4.0 to 5.0 since 1963, M5.0 to 6.0 since 1930, and M6.0 and larger since 1850. For the first two categories, completeness time was derived from plots of cumulative number of events versus time. M3 events were not used in the WUS hazard calculations since they are only complete since about 1976 for most areas and may not even be complete after 1976 for some areas. For California M4.0 to M5.0 since 1933, M5.0 to 6.0 since 1900, and M6.0 and larger since 1850

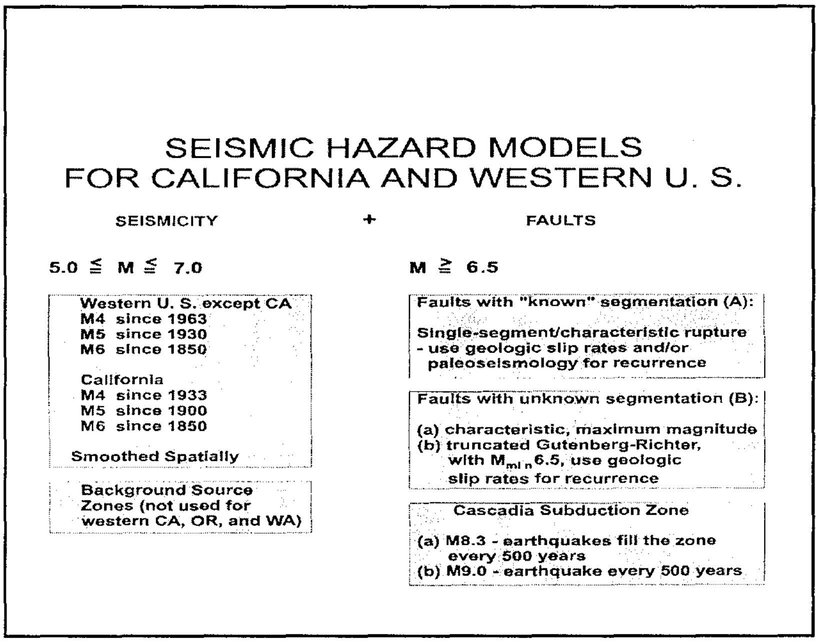


FIGURE B6 Seismic hazard models for California and the western United States. Smoothed seismicity models are shown on the left and fault models are shown on the right.

were used. The catalog for California is complete to earlier dates compared to the catalogs for the rest of the WUS (see below).

Another difference with the CEUS is that multiple models with different minimum magnitudes for the a-value estimates (such as models 1-3 for the CEUS) were not used. The use of such multiple models in the CEUS was partially motivated by the observation that some m_b4 and m_b5 events in the CEUS occurred in areas with few m_b3 events since 1924 (e.g., Nemaha Ridge events and western Minnesota events). It was considered desirable to be able to give such m_b4 and m_b5 events extra weight in the hazard calculation over what they would have in one run with a minimum magnitude of 3. In contrast it appears that virtually all M5 and M6 events in the WUS have occurred in areas with numerous M4 events since 1965. There was also reluctance to use a WUS model with a-values based on a minimum magnitude of 6.0, since this would tend to double count events that have occurred on mapped faults included in Figure B6 right.

For model 1, the gridded a-values were smoothed with a Gaussian with a correlation distance of 50 km, as in model 1 for the CEUS. The hazard calculation from the gridded a-values differed from that in the CEUS, because we considered fault finiteness in the WUS calculations. For each source grid cell, a fictitious fault for magnitudes of 6.0 and larger was used. The fault was centered on the center of the grid cell. The strike of the fault was random and was varied for each magnitude increment. The length of the fault was determined from the relations of Wells and Coppersmith (1994). The fictitious faults were taken to be vertical.

A maximum moment magnitude of 7.0 was used for models 1 and 2, except for four shear zones in northeastern California and western Nevada described below. Of course, larger moment magnitudes are included in the specific faults. A minimum moment magnitude of 5.0 were used for models 1 and 2. For each WUS site, the hazard calculation was done for source-site distances of 200 km and less, except for the Cascadia subduction zone, where the maximum distance was 1000 km.

Separate hazard calculations for deep events (> 35 km) were done. These events were culled from the catalogs. Their a-values were calculated separately from the shallow events. Different attenuation relations were used.

Regional b-values were calculated based on the method of Weichert (1980), using events with magnitudes of 4 and larger and using varying completeness times for different magnitudes. Accordingly, a regional b-value of 0.8 was used in models 1 and 2 for the WUS runs based on shallow events. For the deep events (>35 km), an average b-value of 0.65 was found. This low b-value was used in the hazard calculations for the deep events.

We used a b-value of 0.9 for most of California, except for the easternmost portion of California in our basin and range background zone (see below). This b-value was derived by CDMG.

Faults

The hazard from about 500 Quaternary faults or fault segments was used for the maps. Faults were considered where geologic slip rates have been determined or estimates of recurrence times have been made from trenching studies. A table of the fault parameters used in the hazard calculations has been compiled and is shown on the USGS Internet Web site. Figure B7 shows the

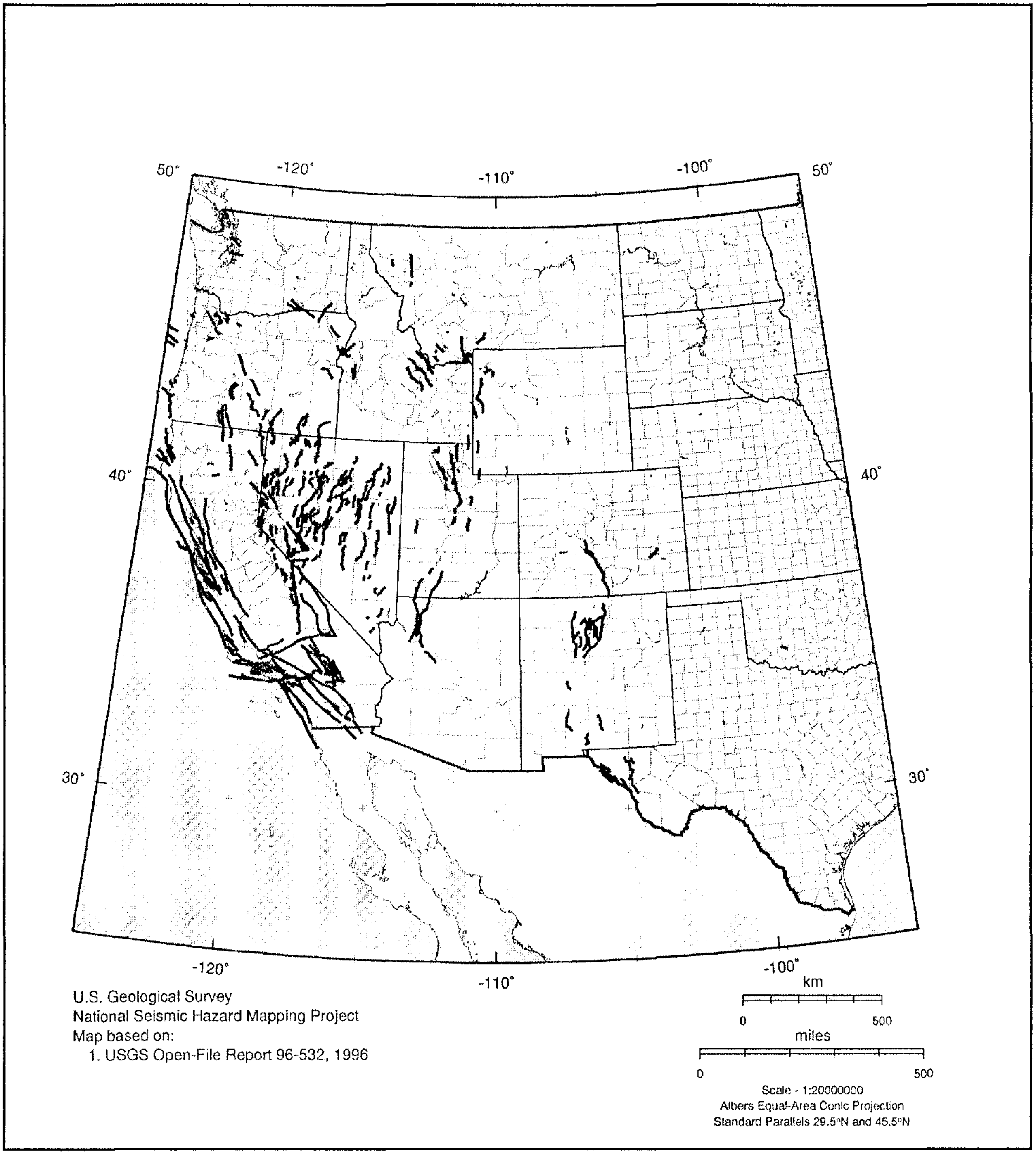


FIGURE B7 Western U.S. faults included in the maps.

faults used in the maps. The numerous individuals who worked on compilations of fault data are too numerous to cite here. They are cited, along with their contribution, in the map documentation (Frankel, et al, 1996).

Recurrence Models for Faults

The hazard from specific faults is added to the hazard from the seismicity as shown in Figure B6. Faults are divided into types A and B, roughly following the nomenclature of WGCEP (1995). A fault is classified as A-type if there have been sufficient studies of it to produce models of fault segmentation. In California the A-type faults are: San Andreas, San Jacinto, Elsinore, Hayward, Rodgers Creek, and Imperial (M. Petersen, C. Cramer, and W. Bryant, written comm., 1996). The only fault outside of California classified as an A-type is the Wasatch Fault. Single-segment ruptures were assumed on the Wasatch Fault.

For California, the rupture scenarios specified by Petersen, Cramer and Bryant of CDMG, with input from Lienkaemper of USGS for northern California were used. Single-segment, characteristic rupture for the San Jacinto and Elsinore faults were assumed. For the San Andreas fault, multiple-segment ruptures were included in the hazard calculation, including repeats of the 1906 and 1857 rupture zones, and a scenario with the southern San Andreas fault rupturing from San Bernardino through the Coachella segment. Both single-segment and double-segment ruptures of the Hayward Fault were included.

For California faults, characteristic magnitudes derived by CDMG from the fault area using the relations in Wells and Coppersmith (1994) were used. For the remainder of the WUS, the characteristic magnitude was determined from the fault length using the relations of Wells and Coppersmith (1994) appropriate for that fault type.

For the B-type faults, it was felt there were insufficient studies to warrant specific segmentation boundaries. For these faults, the scheme of Petersen et al. (1996)was followed, using both characteristic and Gutenberg-Richter (GR; exponential) models of earthquake occurrence. These recurrence models were weighted equally. The G-R model basically accounts for the possibility that a fault is segmented and may rupture only part of its length. It was assumed that the G-R distribution applies from a minimum moment magnitude of 6.5 up to a moment magnitude corresponding to rupture of the entire fault length.

The procedure for calculating hazard using the G-R model involves looping through magnitude increments. For each magnitude a rupture length is calculated using Wells and Coppersmith (1994). Then a rupture zone of this length is floated along the fault trace. For each site, the appropriate distance to the floating ruptures is found and the frequency of exceedance (FE) is calculated. The FE's are then added for all the floating rupture zones.

As used by USGS, the characteristic earthquake model (Schwartz and Coppersmith, 1984) is actually the maximum magnitude model of Wesnousky (1986) Here it is assumed that the fault only generates earthquakes that rupture the entire fault. Smaller events along the fault would be incorporated by models 1 and 2 with the distributed seismicity or by the G-R model described above.

It should be noted that using the G-R model generally produces higher probabilistic ground motions than the characteristic earthquake model, because of the more frequent occurrence of earthquakes with magnitudes of about 6.5.

Fault widths (except for California)were determined by assuming a seismogenic depth of 15 km and then using the dip, so that the width equaled 15 km divided by the sine of the dip. For most normal faults a dip of 60 degrees is assumed. Dip directions were taken from the literature. For the Wasatch, Lost River, Beaverhead, Lemhi, and Hebgen Lake faults, the dip angles were taken from the literature (see fault parameter table on Web site). Strike-slip faults were assigned a dip of 90 degrees. For California faults, widths were often defined using the depth of seismicity (J. Lienkaemper, written comm., 1996; M. Petersen, C. Cramer, and W. Bryant, written comm., 1996). Fault length was calculated from the total length of the digitized fault trace.

Special Cases

There are a number of special cases which need to be described.

Blind thrusts in the Los Angeles area: Following Petersen et al (1996) and as discussed at the Pasadena workshop, 0.5 weight was assigned to blind thrusts in the L.A. region, because of the uncertainty in their slip rates and in whether they were indeed seismically active. These faults are the Elysian Park thrust and the Compton thrust. The Santa Barbara Channel thrust (Shaw and Suppe, 1994) also has partial weight, based on the weighting scheme developed by CDMG.

Offshore faults in Oregon: A weight of 0.05 was assigned to three offshore faults in Oregon identified by Goldfinger et al. (in press) and tabulated by Geomatrix (1995): the Wecoma, Daisy Bank and Alvin Canyon faults. It was felt the uncertainty in the seismic activity of these faults warranted a low weight, and the 0.05 probability of activity decided in Geomatrix (1995) was used. A 0.5 weight was assigned to the Cape Blanco blind thrust.

Lost River, Lemhi and Beaverhead faults in Idaho: It was assumed that the magnitude of the Borah Peak event (M7.0) represented a maximum magnitude for these faults. As with (3), the characteristic model floated a M7.0 along each fault. The G-R model considered magnitudes between 6.5 and 7.0. Note that using a larger maximum magnitude would lower the probabilistic ground motions, because it would increase the recurrence time.

Hurricane and Sevier-Torroweap Faults in Utah and Arizona: The long lengths of these faults (about 250 km) implied a maximum magnitude too large compared to historical events in the region. Therefore a maximum magnitude of M7.5 was chosen. The characteristic and G-R models were implemented as in case (3). Other faults (outside of California) where the M_{max} was determined to be greater than 7.5 based on the fault length were assigned a maximum magnitude of 7.5.

Wasatch Fault in Utah: Recurrence times derived from dates of paleoearthquakes by Black et al. (1995) and the compilation of McCalpin and Nishenko (1996) were used

Hebgen Lake Fault in Montana: A characteristic moment magnitude of 7.3 based on the 1959 event (Doser, 1985) was used.

Short faults: All short faults with characteristic magnitudes of less than 6.5 were treated with the characteristic recurrence model only (weight=1). No G-R relation was used. If a fault had a characteristic magnitude less than 6.0, it was not used.

Seattle Fault: The characteristic recurrence time was fixed at 5000 years, which is the minimum recurrence time apparent from paleoseismology (R. Bucknam, pers. comm., 1996). Using the characteristic magnitude of 7.1 derived from the length and a 0.5 mm/yr slip rate yielded a characteristic recurrence time of about 3000 years.

Eglington fault near Las Vegas: The recurrence time for this fault was fixed at 14,000 years, similar to the recurrence noted in Wyman et al. (1993).

Shear Zones in Eastern California and Western Nevada: Areal shear zones were added along the western border of Nevada extending from the northern end of the Death Valley fault through the Tahoe-Reno area through northeast California ending at the latitude of Klamath Falls, Oregon. A shear rate of 4 mm/yr to zone 1, and 2 mm/yr to zones 2 and 3 was assigned. The shear rate in zone 1 is comparable to the shear rate observed on the Death Valley fault, but which is not observed in mapped faults north of the Death Valley fault (C. dePolo and J. Anderson, pers. comm., 1996). For the Foothills Fault system (zone 4) a shear rate of 0.05 mm/yr was used. a-values were determined for these zones in the manner described in Ward(1994). For zones 1-3, a magnitude range of 6.5-7.3 was used. For zone 4, a magnitude range of 6.0-7 was used. The maximum magnitude for the calculation of hazard from the smoothed historic seismicity was lowered in these zones so that it did not overlap with these magnitude ranges. Fictitious faults with a fixed strike were used in the hazard calculation for these zones. Again, use of these areal zones in California was agreed upon after consultation with CDMG personnel.

Cascadia Subduction Zone

Two alternative scenarios for great earthquakes on the Cascadia subduction zone were considered. For both scenarios it was assumed that the recurrence time of rupture at any point along the subduction zone was 500 years. This time is in or near most of the average intervals estimated from coastal and offshore evidence (see Atwater and Hemphill-Haley, 1996; Geomatrix, 1995; B. Atwater, written comm., 1996). Individual intervals, however, range from a few hundred years to about 1000 years (Atwater et al., 1995).

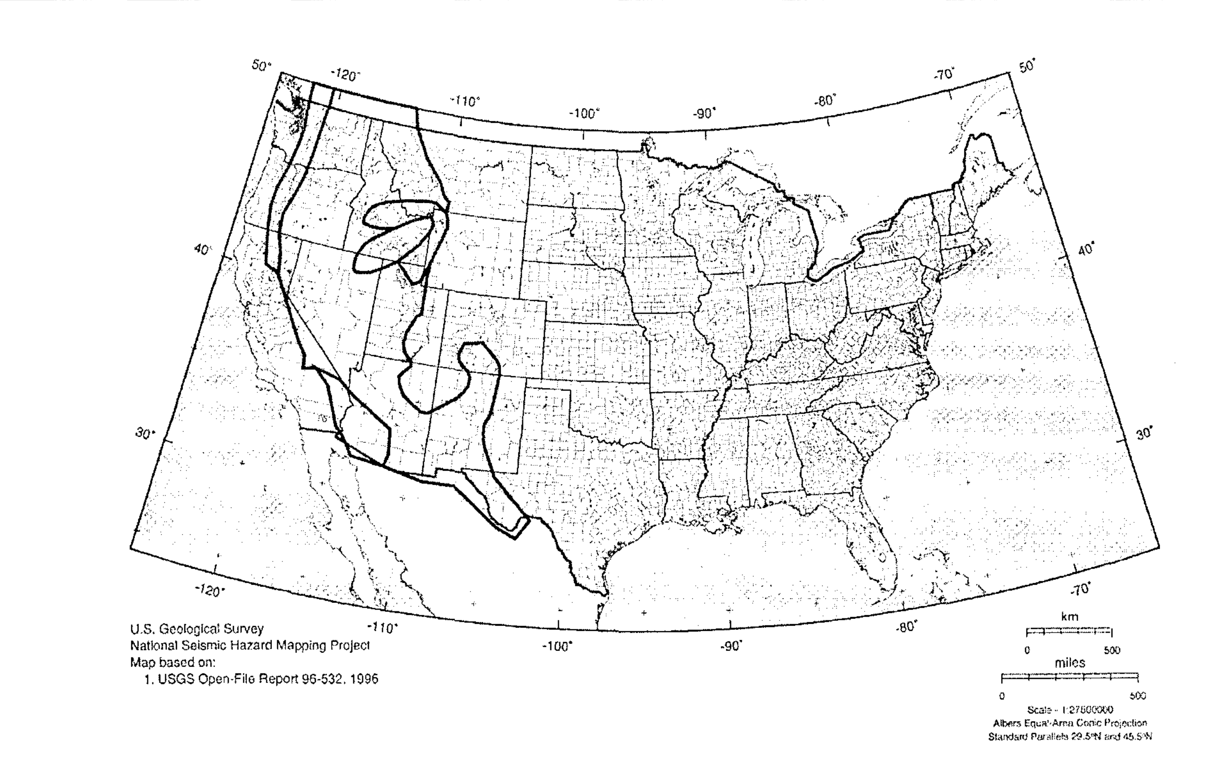
The first scenario is for moment magnitude 8.3 earthquakes to fill the subduction zone every 500 years. Based on a rupture length of 250 km (see Geomatrix, 1995) for an M8.3 event and the 1100 km length of the entire subduction zone, this requires a repeat time of about 110 years for an M8.3 event. However, no such event has been observed in the historic record of about 150 years. This M8.3 scenario is similar to what was used in the 1994 edition of the USGS maps (see Leyendecker et al., 1995) and it is comparable to the highest weighted scenario in Geomatrix (1995). A M8.3 rupture zone was floated along the strike of the subduction zone to calculate the hazard. A weight of 0.67 was assigned for this scenario in the maps.

The second scenario used is for a moment magnitude 9.0 earthquake to rupture the entire Cascadia subduction zone every 500 years on average. No compelling reason was seen to rule out such a scenario. This scenario would explain the lack of M8s in the historic record. It is also consistent with a recent interpretation of Japanese tsunami records by Satake et al. (1996). By ruling out alternative source regions, Satake et al. (1996) reported that a tsunami in 1700 could have been produced by a M9.0 earthquake along the Cascadia subduction zone. A weight of 0.33 was assigned to the M9.0 scenario in the maps.

The subduction zone was specified as a dipping plane striking north-south from about Cape Mendocino to 50 degrees north. It was assumed that the plane reached 20 km depth at a longitude of 123.8 degrees west, just east of the coastline. This corresponds roughly to the 20 km depth contour drawn by Hyndman and Wang (1995) and is consistent with the depth and location of the Petrolia earthquake in northern California. A dip of 10 degrees was assigned to the plane and a width of 90 km. The seismogenic portion of the plane was assumed to extend to a depth of 20 km.

Background Source Zones

The background source zones for the WUS (model 2) were based on broad geologic criteria and were developed by discussion at the Salt Lake City (SLC) workshop (except for the Cascades source zone). These zones are shown in Figure B8. Note that there are no background source zones west of the Cascades and west of the Basin and Range province. For those areas, model 1 was used with a weight of 1.



FUGURE B8 Western U.S. background zones.

At the SLC workshop there was substantial sentiment for a Yellowstone Parabola source zone (see, e.g., Anders et al., 1989) that would join up seismically-active areas in western Wyoming with the source areas of the Bora Peak and Hebgen Lake earthquakes. It was felt that the relatively seismically-quiescent areas consisting of the Snake River Plain and Colorado Plateau should be separate source zones because of the geologic characteristics. An area of southwest Arizona was suggested as a separate source zone by Bruce Scheol, based partly on differences in the age and length of geologic structures compared with the Basin and Range Province (see Edge et al., 1992). A Cascades source zone was added since it was felt that was a geologically-distinct area.

The remaining background source zone includes the Basin and Range Province, the Rio Grande Rift, areas of Arizona and New Mexico, portions of west Texas, and areas of eastern Washington and northern Idaho and Montana. The northern border of this zone follows the international border. As stated above, this seems to be a valid approach since the hazard maps are being based on the seismicity rate in the area of interest.

This large background zone is intended to address the possibility of having large earthquakes (M6 and larger) in areas with relatively low rates of seismicity in the brief historic record. It is important to have a large zone that contains areas of high seismicity in order to quantify the hazard in relatively quiescent areas such as eastern Oregon and Washington, central Arizona, parts of New Mexico, and west Texas. One can see the effect of this large background zone by noting the contours on the hazard maps in these areas. The prominence of the background zones in the maps is determined by the weighting of models 1 and 2.

Adaptive Weighting for the WUS

The adaptive weighting procedure was used to include the background zones in the WUS without lowering the hazard values in the high seismicity areas. As with the CEUS, the a-value was checked for each source cell to see whether the rate from the historic seismicity exceeded that from the appropriate background zone. If it did, the a-value was used from the historic seismicity. If the historic seismicity a-value was below the background value, then a rate derived from using 0.67 times the historic rate plus 0.33 times the background rate was used. This does not lower the a-value in any cell lower than the value from the historic seismicity. The total seismicity rate in this portion of the WUS in the new a-value grid is 16 percent above the historic rate (derived from M4 and greater events since 1963).

WUS Catalogs

For the WUS, except for California, the Stover and Coffman (1993), Stover, Reagor, and Algermissen (1984), PDE, and DNAG catalogs (with the addition of Alan Sanford's catalog for New Mexico) were used. For California, a catalog compiled by Mark Petersen of California Division of Mines and Geology (CDMG) was used. Mueller et al. (1996) describes the processing of the catalogs, the removal of aftershocks, and the assignment of magnitudes. Utah coal-mining events were removed from the catalog (see Mueller et al., 1996). Explosions at NTS and their aftershocks were also removed from the catalog.

Attenuation Relations for WUS

Crustal Events: For spectral response acceleration, three equally-weighted attenuation relations were used: (1) Boore, Joyner, and Fumal (BJF; 1993, 1994a) with later modifications to differentiate thrust and strike-slip faulting (Boore et al., 1994b) and (2) Sadigh et al. (1993). For (1) ground motions were calculated for a site with average shear-wave velocity of 760 m/sec in the top 30m, using the relations between shear-wave velocity and site amplification in Boore et al. (1994a). For (2) their "rock" values were used. Joyner (1995) reported velocity profiles compiled by W. Silva and by D. Boore showing that WUS rock sites basically spanned the NEHRP B/C boundary. When calculating ground motions for each fault, the relations appropriate for that fault type (e.g, thrust) were used. All of the relations found higher ground motions for thrust faults compared with strike slip faults.

All calculations included the variability of ground motions. For 1) the sigma values reported in BJF (1994b) were used. For 2) the magnitude-dependent sigmas found in those studies were used.

The distance measure from fault to site varies with the attenuation relation and this was accounted for in the hazard codes (see B.5 for additional detail on distance measures).

Deep events (> 35 km): Most of these events occurred beneath the Puget Sound region. although some were in northwestern California. For these deep events, only one attenuation relation was used -- i.e., by Geomatrix (1993; with recent modification for depth dependence provided by R. Youngs, written comm., 1996) which is based on empirical data of deep events recorded on rock sites. The relations of Crouse (1991) were used because they were for soil sites. It was found that the ground motions from Geomatrix (1993) are somewhat smaller than those from Crouse (1991), by an amount consistent with soil amplification. These events were placed at a depth of 40 km for calculation of ground motions.

Cascadia subduction zone: For M8.3 events on the subduction zone, two attenuation relations (with equal weights) were used following the lead of Geomatrix (1993): 1) Sadigh et al. (1993) for crustal thrust earthquakes and 2) Geomatrix (1993) for interface earthquakes. For the M9.0 scenario, Sadigh et al. (1993) formulas could not be used since they are invalid over M8.5. Therefore, only Geomatrix (1993) was used. Again the values from Geomatrix (1993) were somewhat smaller than the soil values in Crouse (1991).

ALASKA

The basic procedure, shown in Figure B9, for constructing the Alaska hazard maps is similar to that previously described for the Western United States. The maps have been completed and both the maps and documentation (USGS, 1998a, 1998b) have been placed on the USGS internet site (http://geohazards.cr.usgs.gov/eq/); printing of the maps is in progress.

Faults

The hazard from nine faults was used for the maps (Figure B10). Faults were included in the map when an estimated slip rate was available. The seismic hazard associated with faults not explicitly included in the map is captured to a large degree by the smoothed seismicity model. Specific details

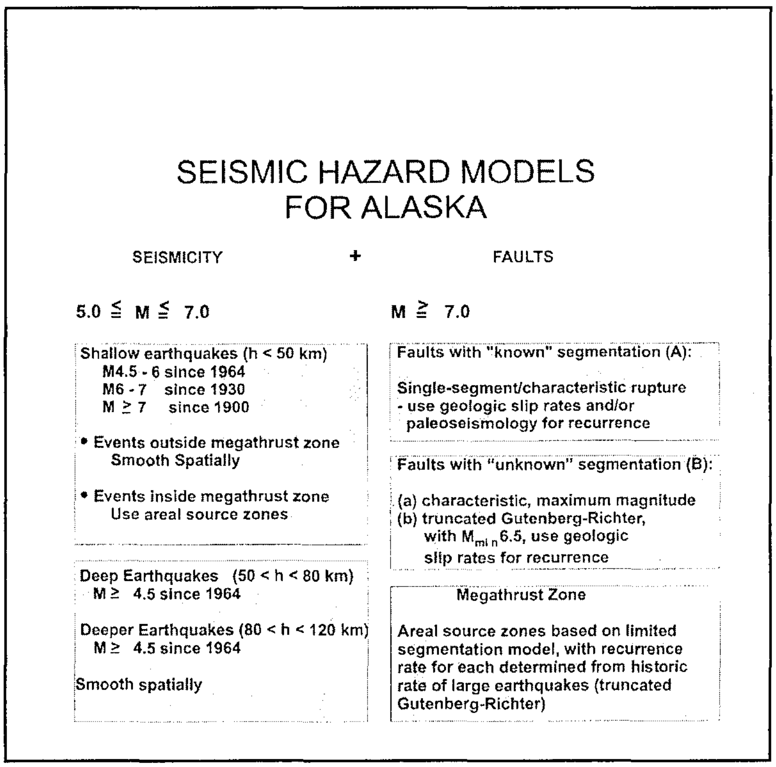


FIGURE B9 Seismic hazard models for Alaska. Smoothed seismicity models are shown on the left and fault models are shown on the right.

on the fault parameters are given in USGS., 1997a. All of the faults except one were strike-slip faults.

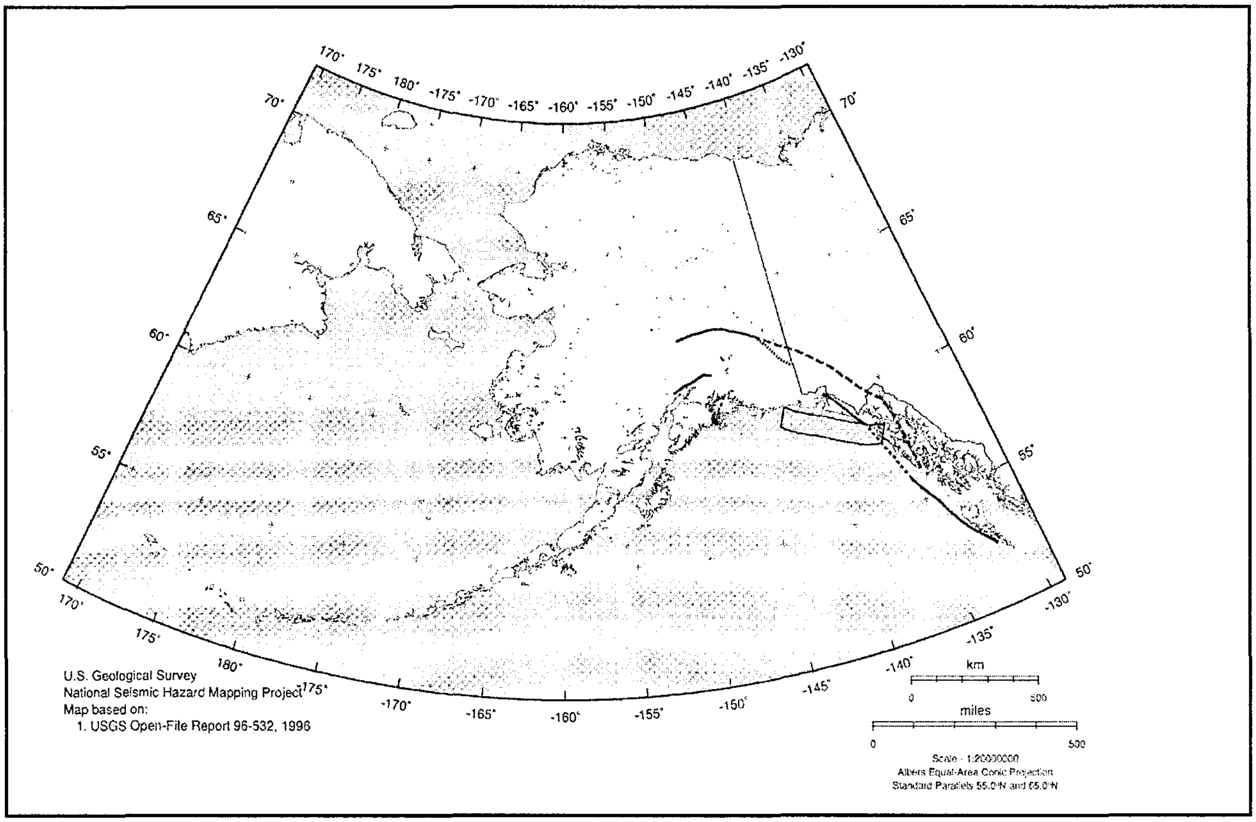


FIGURE B10 Faults included in the maps. Faults are shown with different line types for clarity. Dipping faults are shown as closed polygons.

Recurrence Models for Faults

As was done for the western U.S., faults were divided into types A and B. The fault treatment was the same as the western U.S. Type A faults were the Queen Charlotte, Fairweather offshore, Fairweather onshore, and Transition fault. Type B faults included western Denali, eastern Denali, Totshunda, and Castle Mountain.

For the type B faults, both characteristic and Gutenberg-Richter (G-R) models of earthquake occurrence were used. These recurrence models were weighted equally. The G-R model accounts for the possibility that a fault is segmented and may rupture only part of its length. It was assumed that the G-R distribution applies from a minimum moment magnitude of 6.5 up to a moment magnitude corresponding to rupture of the entire fault length.

Special Case

The Transition fault was treated as a Type A fault even though its segmentation is unknown. Although the rationale for this treatment is documented in USGS, 1998a, it should be pointed out that the parameters, such as segmentation and slip rate, associated with this fault are highly uncertain.

Megathrust

The Alaska-Aleutian megathrust was considered in four parts, shown in Figure B11. Specific rationale for the use of these boundaries is complex and is described in USGS, 1998a.

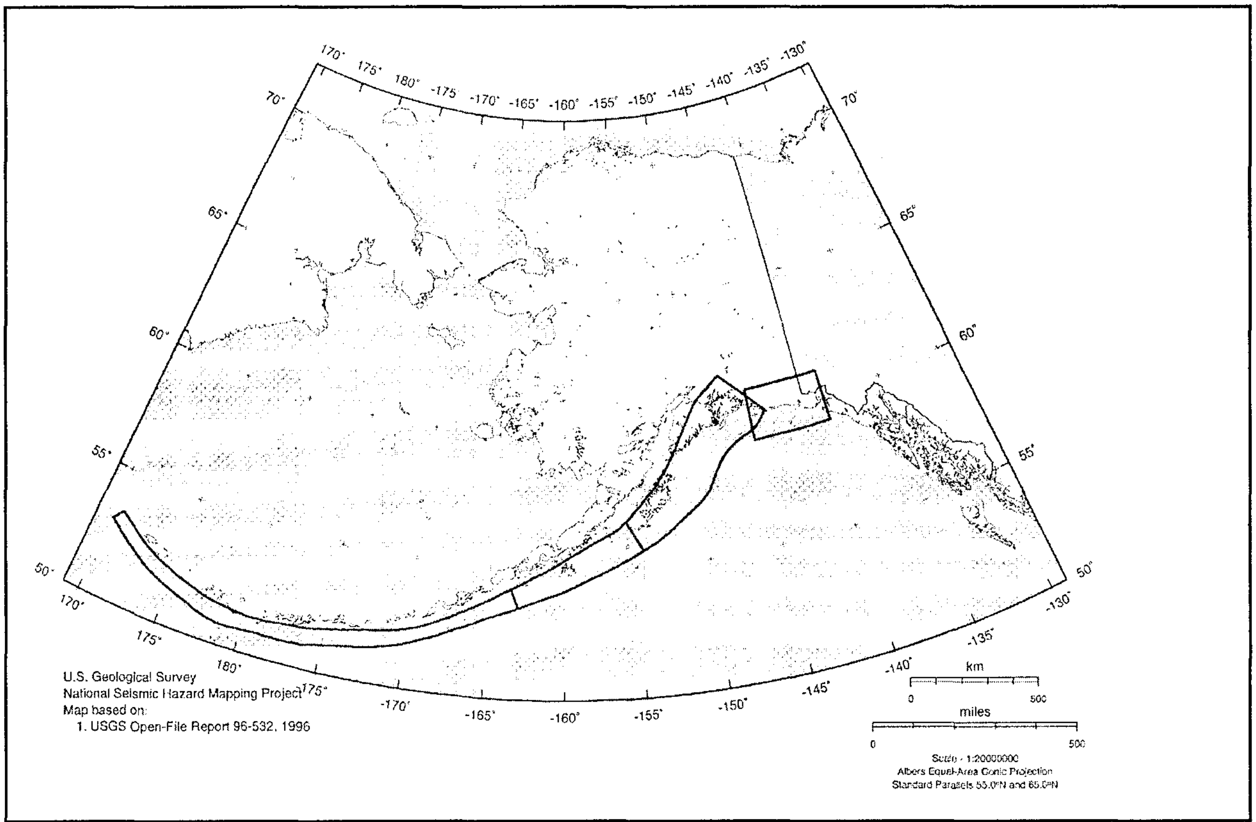


FIGURE B11 Subduction zones included in the maps.

Alaska Catalogs

A new earthquake catalog was built by combining Preliminary Determination of Epicenter, Decade of North American Geology, and International Seismological Centre catalogs with USGS interpretations of catalog reliability. Mueller et al. (1997) describes the processing of the catalogs, the removal of aftershocks, and the assignment of magnitudes.

Attenuation Relations for Alaska

Crustal Events: For spectral response acceleration, two equally-weighted attenuation relations were used: (1) Boore, Joyner, and Fumal (BJF; 1997) and (2) Sadigh et al. (1997). For (1) ground motions were calculated for a site with average shear-wave velocity of 760 m/sec in the top 30m. For (2) their "rock" values were used. These are recent publication of the attenuations cited for the western U.S. The attenuations are the same. When calculating ground motions for each fault, the relations appropriate for that fault type (e.g, strike slip) were used. All calculations included the variability of ground motions.

Deep events (50 - 80 km): For these deep events, only one attenuation relation was used, the intraslab form of Youngs et al (1997) with a depth fixed at 60 km.

Deeper events (80 - 120 km): For these deeper events, only one attenuation relation was used, the intraslab form of Youngs et al (1997) with a depth fixed at 90 km.

Megathrust and Transition Fault: Only one attenuation relation was used, the interslab form of Youngs et al (1997). It should be noted that the use of this attenuation for the Transition fault resulted in lower ground motions than would have been obtained using the crustal attenuation equations.

PROBABILISTIC MAPS

Two of the probabilistic maps were key to the decisions made by the SDPG for developing the *maximum considered earthquake* ground motion maps. These are the 0.2 sec and 1.0 sec spectral response maps for a 2 percent probability of exceedance in 50 years. These are shown in Figures 12 and 13 respectively. The way in which these maps were used is described in the following sections.

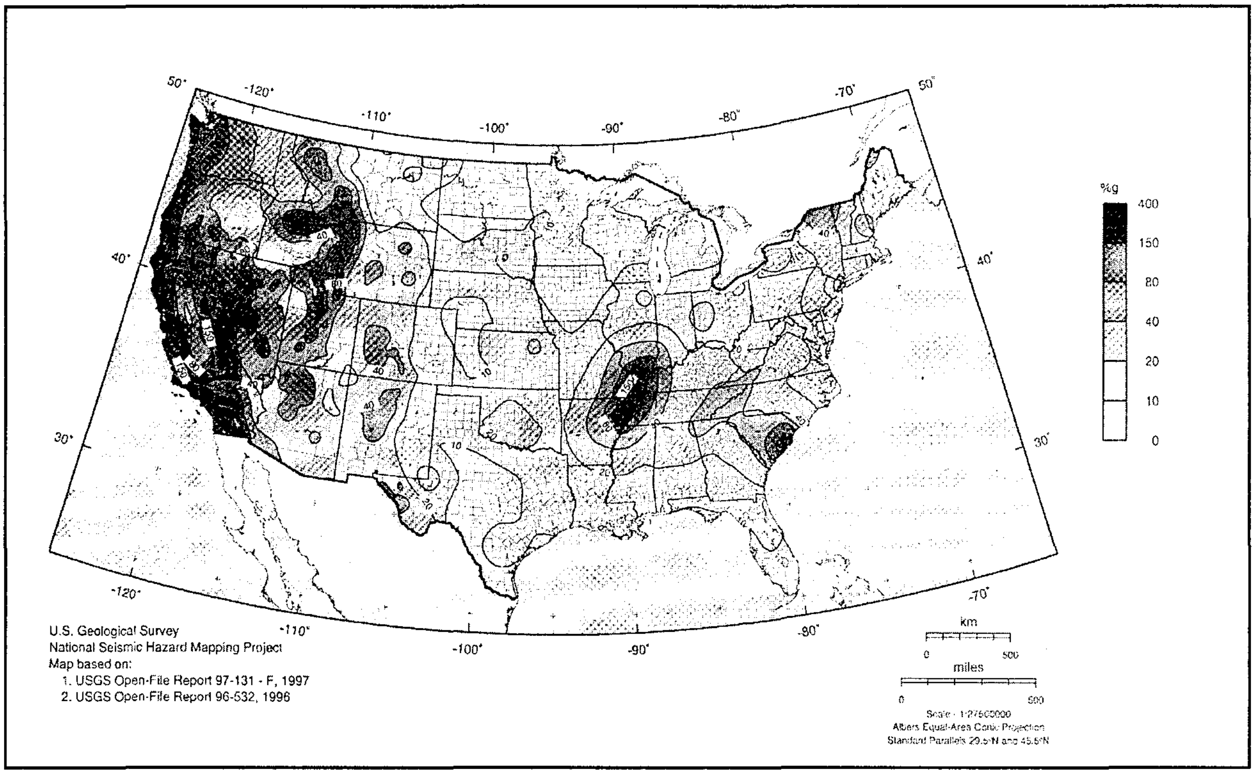


FIGURE B12 Probabilistic map of 0.2 sec spectral response acceleration with a 2% probability of exceedance in 50 years. The reference site material has a shear wave velocity of 750 m/sec.

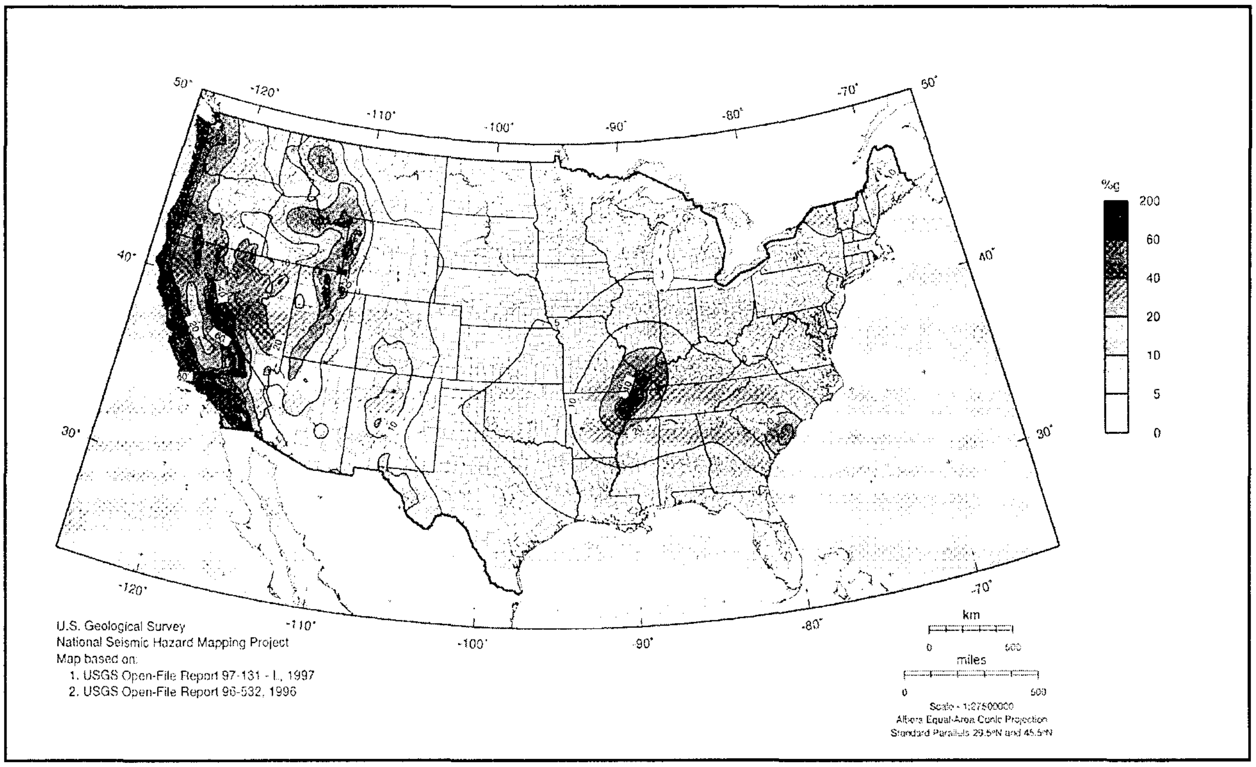


FIGURE B13 Probabilistic map of 1.0 sec spectral response acceleration with a 2% probability of exceedance in 50 years. The reference site material has a shear wave velocity of 750 m/sec.

DEVELOPMENT OF NEHRP MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION MAPS

The *maximum considered earthquake* spectral acceleration maps were derived from the 2 percent in 50 year probabilistic maps shown simplified as Figures 12 and 13 (also see Frankel, et al, 1997), discussed above, with the application of the SDPG rules also described previously. Additional detail in applying the rules is described in this section. The 0.2 sec map is used for illustration purposes. The same procedures and similar comments apply for the 1.0 sec map.

One of the essential features of the SDPG rules was that the recommendations, when applied by others, would result in the same maps. This procedures allows the use of engineering judgement to be used in developing the maps, as long as those judgements are explicitly stated. This approach will simplify modification of the recommendations as knowledge improves.

It should be noted that although the maps are termed *maximum considered earthquake* Ground Motion maps. These maps are not for a single earthquake. The maps include probabilistic effects which consider all possible earthquakes up to the plateau level. Above the plateau level, the contours are included for the deterministic earthquake on each fault (unless the deterministic value is higher than the probabilistic values).

Deterministic Contours

The deterministic contours, when included, are computed using the same attenuation functions used in the probabilistic analysis. However, the deterministic values are not used unless they are less than the probabilistic values. After study of those areas where the plateau was reached, the only areas where the deterministic values were less than the probabilistic values were located in California and along the subduction zone region of Washington and Oregon. Further study indicated that those areas with values in excess of the plateau were located in California. The appropriate attenuation for this area were the Boore-Joyner-Fumal attenuation (1993,1994) and the Sadigh et al (1993) attenuation.

The form of these attenuations and the distance measures used have an effect on the shape of these deterministic contours. Accordingly, they are discussed below. The Boore-Joyner-Fumal equation is:

$$logY = b_{ss}G_{ss} + b_{RS}G_{RS} + b_2(M - 6)2 + b_4 r + b_5 log(r) + b_v(logV_s + logV_a)$$

where:

Y	=	ground motion parameter
M	=	earthquake magnitude
b_{SS}, b_{RS}	=	coefficients for strike-slip and reverse-slip faults, determined by regression and different for each ground motion parameter
G_{SS}	=	1.0 for strike-slip fault, otherwise zero
G_{RS}	=	1.0 for reverse-slip fault, otherwise zero
b_2, b_3, b_4, b_5	=	coefficients determined by regression, different for each spectral acceleration
b_V	=	coefficient determined by regression, different for each spectral acceleration
V_A	=	coefficient determined by regression, different for each spectral acceleration
V_S	=	shear wave velocity for different site category
r	=	$(d^2 + h^2)^{1/2}$
d	=	closest horizontal distance from the site of interest to the surface projection of the rupture surface, see Figure B14
h	=	fictitious depth determined by regression, different for each ground motion parameter

Coefficients determined by regression are tabulated in the reports describing the attenuation equation.

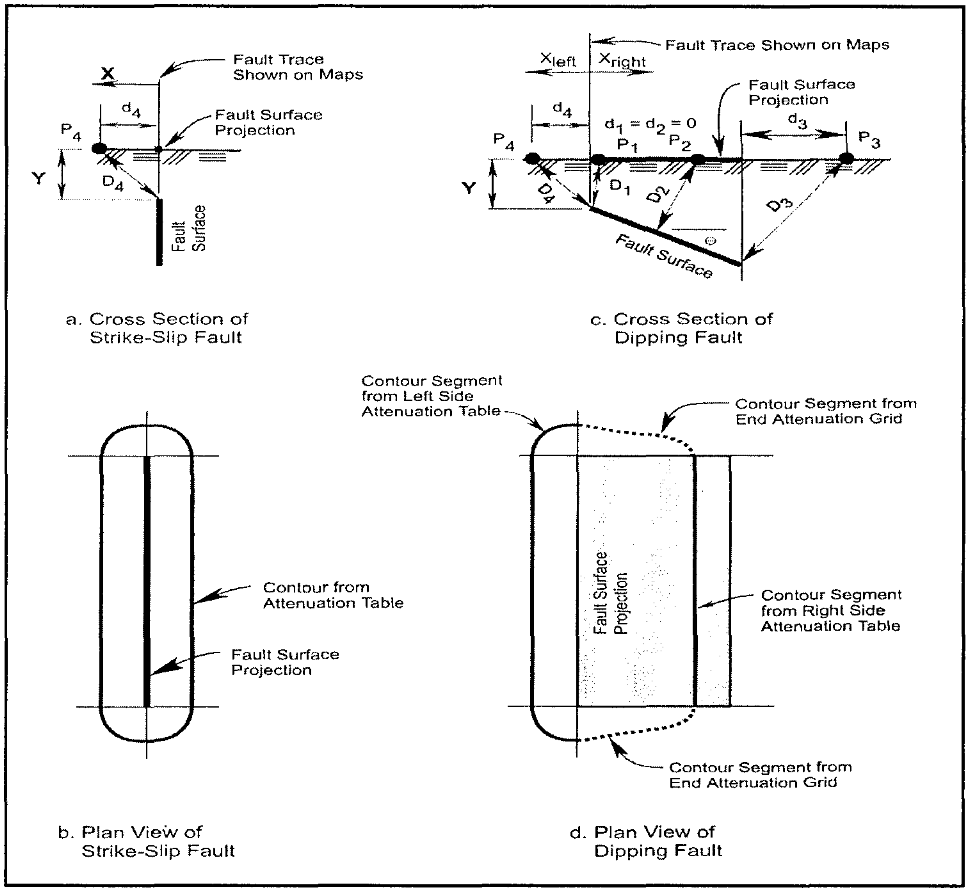


FIGURE B14 Measures of distance for strike-slip and dipping faults. A cross section of strike-slip fault is shown in figure (a) and the shape of a typical deterministic contour is shown in figure (b). A dipping fault is shown in figure (c) and the shape of a typical deterministic contour is shown in figure (d).

The Sadigh et al. equation is:

$$\ln Y(T) = F\{C_1 + C_2M + C_3(8 - =5 - M)^{2.5} + C_4\ln[D + \exp(C_5 + C_6M)] + C_y\ln(D + 2)\}$$

where:

Y	=	spectral response acceleration at period T
M	=	earthquake magnitude
$C_1, C_2, C_3 \ldots C_7$	=	coefficients determined by regression, different for each ground motion parameter
D	=	closest distance to the fault rupture surface, see Figure B14
F	=	Factor for fault type. 1.0 for strike-slip faults, 1.2 for reverse/thrust faulting, 1.09 for oblique faults

The distance measures are shown in Figure B14 and are discussed in more detail below.

The computation of spectral response (or any ground motion parameter) is a relatively simple matter for a specific site (or specific distance from a fault) but can become complex when preparing contours since it is difficult to calculate the specific distance at which a particular ground motion occurs This is due to the complexity of the two attenuation functions and the need to combine their results. Since the attenuation functions were weighted equally, each contributes equally to the ground motion at a site. Deterministic contours were determined by preparing attenuation tables, that is the spectral response was computed at various distances from the fault or the fault ends for each earthquake magnitude. Contours for specific values were then drawn by selecting the table for the appropriate magnitude and determining, using interpolation, the distance from the fault for a given spectral acceleration. This procedure required, as a minimum, one attenuation table for each fault. Depending on the fault geometry, more than one table was needed. In order to illustrate this the strike-slip fault is discussed first, followed by a discussion of dipping faults.

Strike-Slip Faults: The strike-slip fault, shown in Figure B14a, b is the simplest introduction to application of the SDPG rules. The distance measures are shown for each attenuation function in Figure B14a. The Boore-Joyner-Fumal equation uses the distance, d_4. The term r in equation includes d_4 and the fictitious depth h. Since h is not zero, $r > d_4$, even if the term y in Figure B14a is zero. The Sadigh et al. equation measures the distance, D, as the closest distance to the rupture surface. In this case to the top of the rupture. If the depth y is zero, then $d_4 = D_4$.

It makes little difference in the computations if the fault rupture plane begins at the surface or at some distance below the surface. For the strike-slip fault the contour for a particular spectral acceleration is a constant distant from the fault and the contour is as shown in Figure B14b. One attenuation table (including the effects of both attenuation equations) can be used for either side of the fault and at the fault ends.

Dipping Faults: The dipping fault, shown in Figures B14c and d, is the most complex case for preparing deterministic contours. The distance measures are shown for each attenuation function in Figure B14c. As before, it is a simple matter to compute the spectral values at a specific site, but not as simple to compute the distance at which a specific spectral acceleration occurs. This is particularly true at the end of the fault.

On the left side of the fault shown in Figure B14c, an attenuation table is prepared, much as in the case of the strike-slip fault. This table may also be used to determine the contour around a portion of the fault end as shown in Figure B14d. In this case it is simply one-quarter of a circle.

A separate attenuation table must be prepared for the right side of the fault as shown in Figure B14d. Since d or D is measured differently, depending on location x, calculations must keep track of whether or not the location is inside or outside of the surface fault projection. Note that the term d is zero when the location x falls within the surface projection, but the fictitious depth h is not. Outside the fault projection, the distance d is measured from the edge of the projection. The distance D is calculated differently, as illustrated in Figure B14c, depending on location but it is always the closest distance to the fault rupture surface.

At the ends of the fault, an attenuation grid was prepared to determine the contour shape shown dotted in Figure B14d. The contour in this area was digitized using the gridded values and combined with the remainder of the contour determined from the left and right attenuation tables.

This need for digitizing a portion of the contour greatly increased the time required to prepare each of the contours for dipping faults. In short, each dipping fault required two attenuation tables and an attenuation grid to prepare each deterministic contour. Thus preparation of each contour is far more time-consuming than preparing a contour for a strike-slip fault. Each contour is unsymmetrical around the fault, the amount of asymmetry depends on the angle of dip.

It can be argued that the knowledge of fault locations and geometry does not warrant this level of effort. However, it was considered necessary in order to follow the concept of repeatability in preparing the maps.

Combining Deterministic Contours: Where two or more faults are nearby, as in Figure B15a, the deterministic contours were merged (depending on amplitudes) as shown in Figure B15b. The merging resulted in the sharp "corners" shown in the figure. Although it can be argued that these intersections should be smoothed, it was believed that maintaining the shape reflected the decision to use deterministic contours.

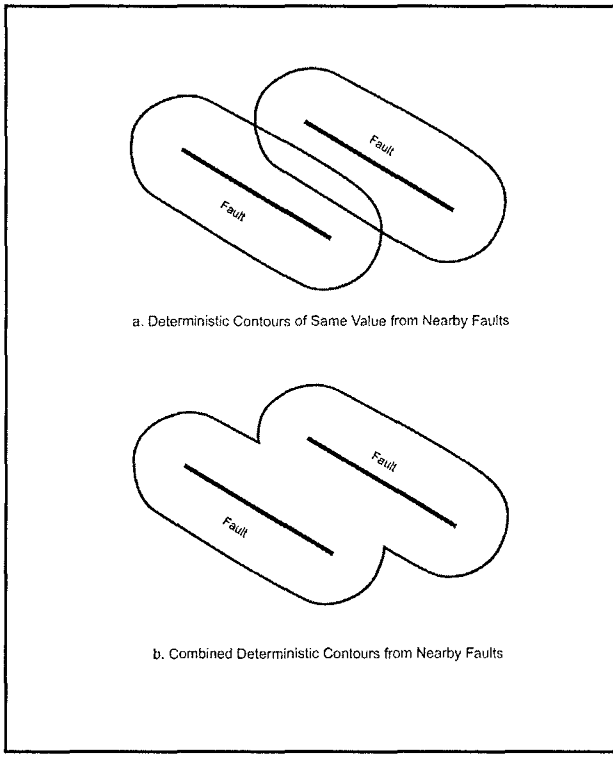


FIGURE B15 Procedure for combining deterministic contours from nearby faults.

Combining Deterministic and Probabilistic Contours

The SDPG decision to use a combination of deterministic and probabilistic contours, although simple in principle, led to number of problems in preparing the contour maps.

Figure B16a, b for a single strike-slip fault illustrates the concept originally envisioned for combining the deterministic and probabilistic contours. After combining the two sets of contours shown in Figure B16a, the *maximum considered earthquake* contours would be as shown in Figure B16b.

In application the situation is more complex, there is frequently more than one fault, with different magnitudes, different return times, different fault geometry, and different locations with respect to each other. Examples are shown in Figures 17 and 18 which will be discussed later. The effect of the variables is illustrated in Figure B16 c and d. The deterministic curve is shown for a single fault with a return time much larger than that of the map. The deterministic spectral acceleration is much larger than the spectral acceleration resulting from historical seismicity. The probabilistic curve is not necessarily symmetrical to the fault. The resulting *maximum considered earthquake* curve shown in Figure B16d is a complex mix of the probabilistic and deterministic curves. There is not always a plateau and the curve is not necessarily symmetrical to the fault, even for a strike-slip fault. Simply stated, the probabilistic curve consider other sources such as historical seismicity and other faults as well as time. The deterministic curve does not consider other sources for this simple example and does not consider time.

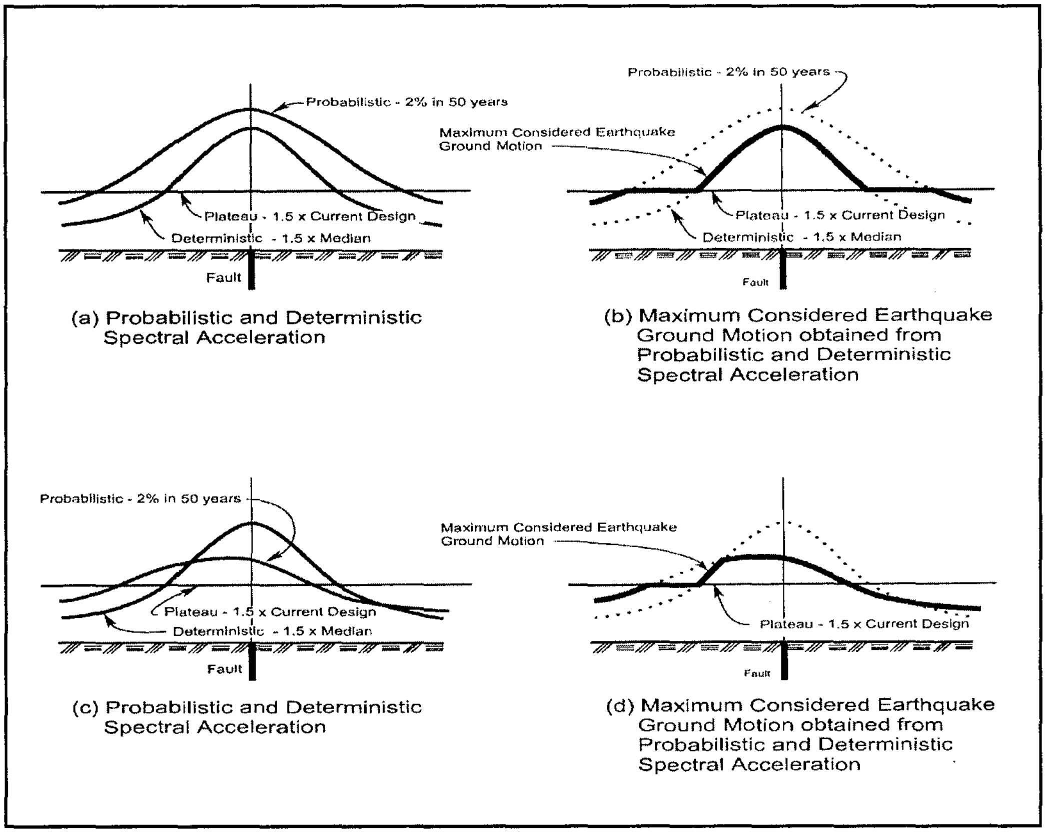


FIGURE B16 Procedure for obtaining maximum considered earthquake ground motion,

The only areas of the United States that have deterministic contours are in California, along the Pacific coast through Oregon and Washington, and in Alaska. At first review it can be seen that there are several other areas that have contours in excess of the plateau but do not have plateaus. In these areas (e.g., New Madrid), the deterministic values exceed the probabilistic ones and thus were not used.

There were several instances where application of the SDPG rules produced results that appear counterintuitive and in other instance produced results that were edited. Two examples from southern California are discussed below. Each example is illustrated with a three-part figure. Part (a) shows both probabilistic contours (dashed) and deterministic contours (solid) for each fault which is also shown. Part (b) shows the *maximum considered earthquake* results produced by following the SDPG rules. Part (c) shows how part (b) was edited for the final map.

Example 1: The first example in Figure B17 illustrates the occurrence of gaps in the deterministic contours around a fault and the halt of a deterministic contour before the end of a fault. When the probabilistic contours and deterministic contours shown in Figure B17a are combined, a gap in the deterministic contours occurs in the vicinity of 34.6^{O} and 118.8^{O}. Similarly the deterministic contours stop prior to the end of the fault around 34.65^{O} and 119.4^{O}. Both of these are shown in Figure B17b.

After study, it is clear that the SDPG rules results in a repeatable, but unusual, set of contours. The result does not go along with the concept of accounting for near fault effects with the deterministic contours. Because of this undesirable effect, the contours were hand edited to restore the gaps and produce the result in Figure B17c.

All occurrences similar to this were edited to modify the contours so that the deterministic contours did not have abrupt breaks or stops before the ends of the fault.

Example 2: The second example in Figure B18 illustrates the occurrence of many faults at different orientations to each other and with different return times. Merging of the complex set of contours is shown in Figure B18b. The contours are greatly simplified. Some small plateaus are shown along the 150 percent contour, as is a gap along one of the faults around 34.0° and 116.35°. The gap was edited as in example 1. The small plateaus were edited out using the judgement that their presence was inconsequential (less than a few percent effect on the maps) and unnecessarily complicated an already complicated map.

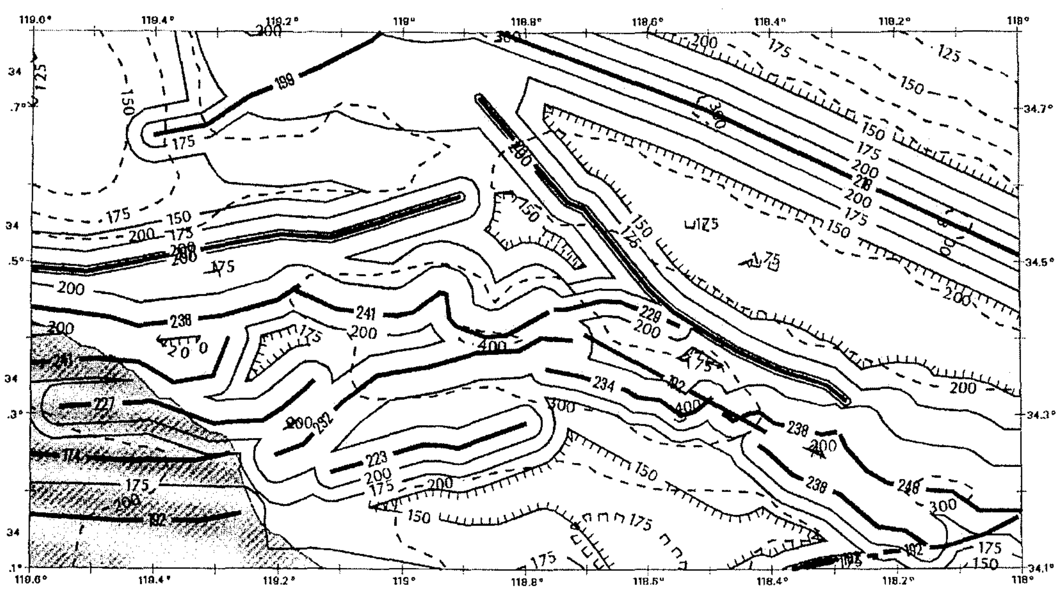


FIGURE B17a Combining contours - Example 1. Both probabilistic and deterministic contours are shown. Probabilistic contours are shown dotted.

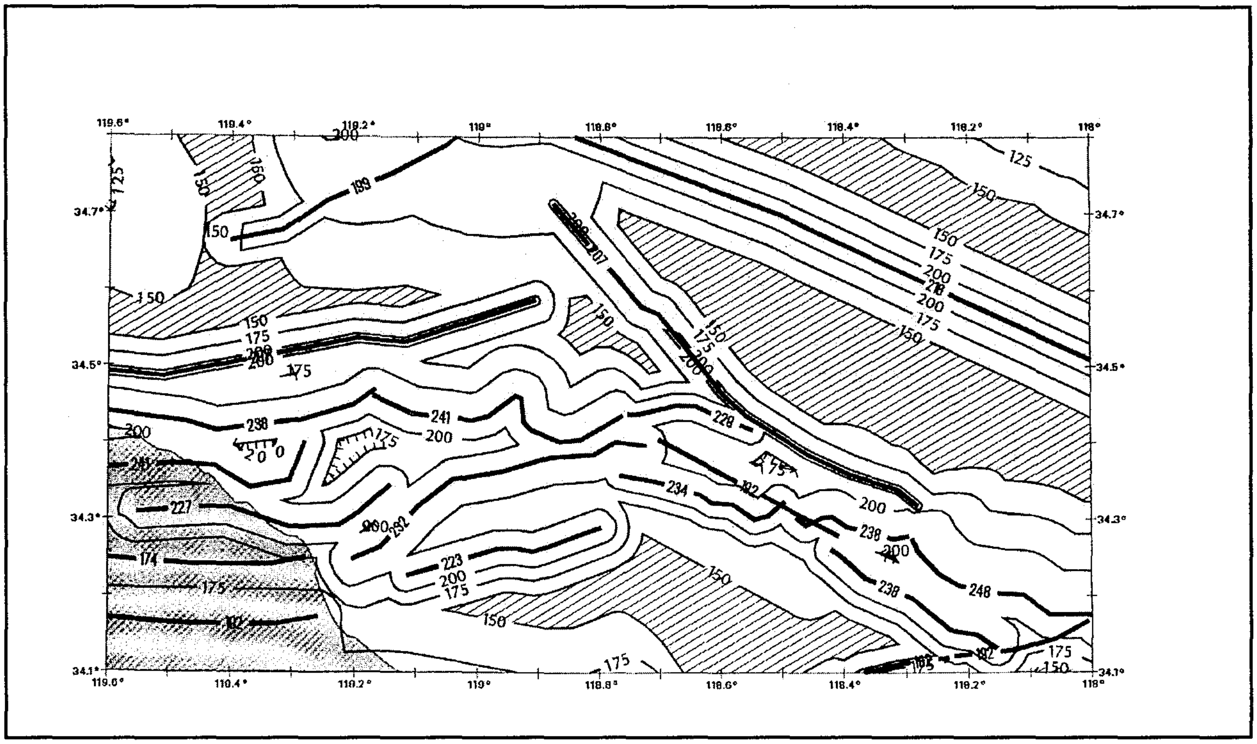


Figure B17b Combining contours - Example 1. Both probabilistic contours are merged using strict interpretation of committee rules.

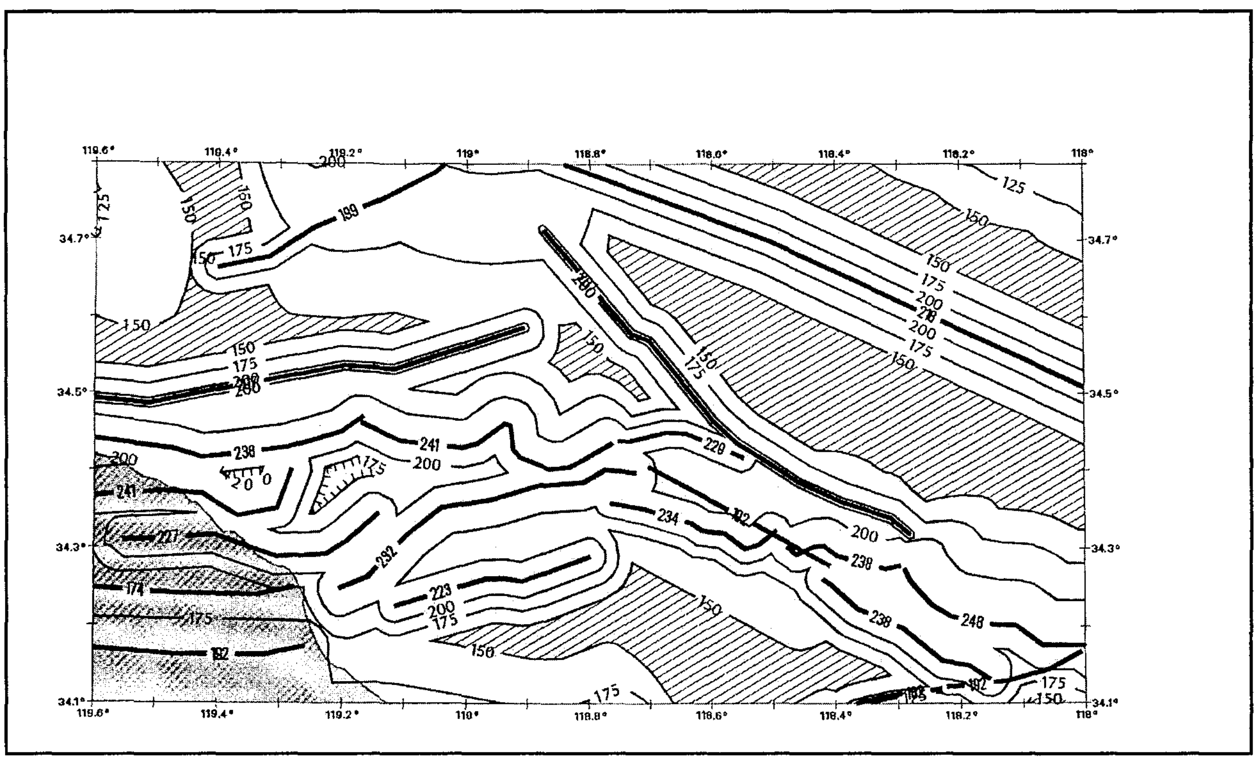


FIGURE B17c Combining contours - Example 1. Probabilistic contours are merged with deterministic contours using strict interpretation of committee rules with subsequent editing.

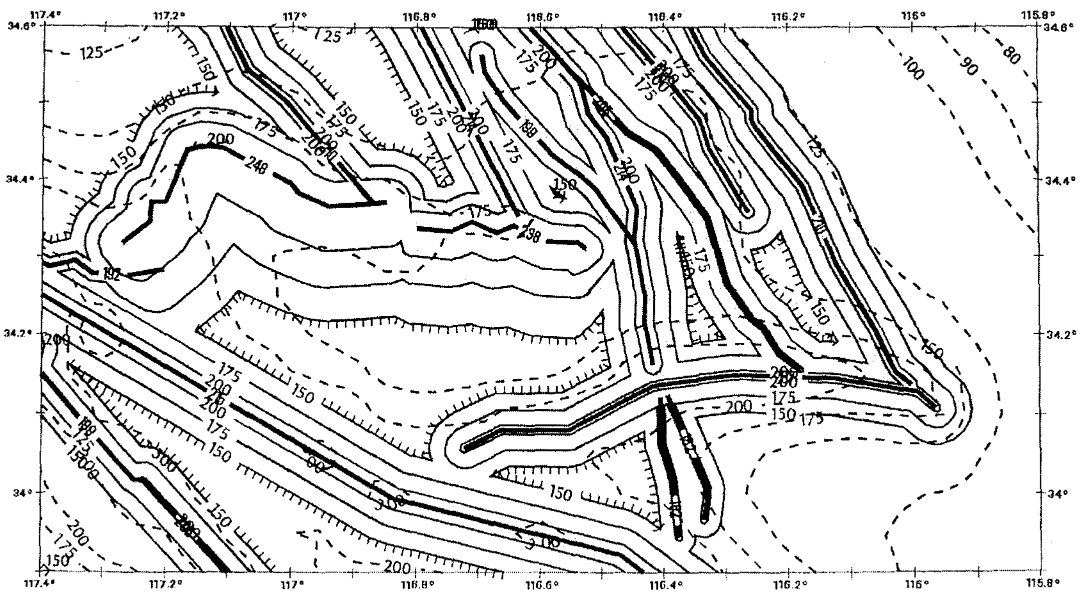


FIGURE B18a Combining contours - Example 1. Both probabilistic and deterministic contours are shown. Probabilistic contours are shown dotted.

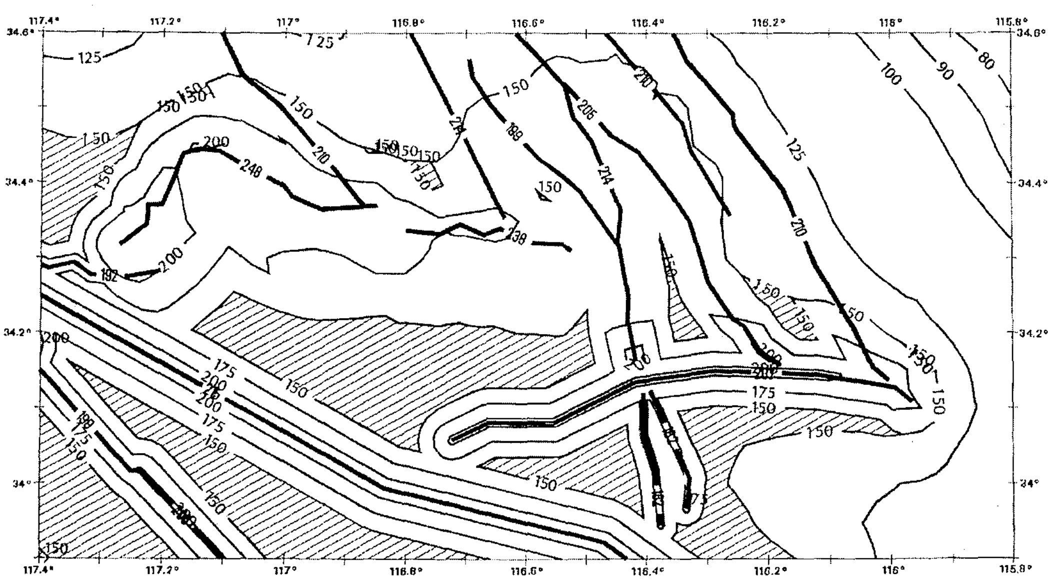


FIGURE B18b Combining contours - Example 1. Probabilistic contours are merged using strict interpretation of committee rules.

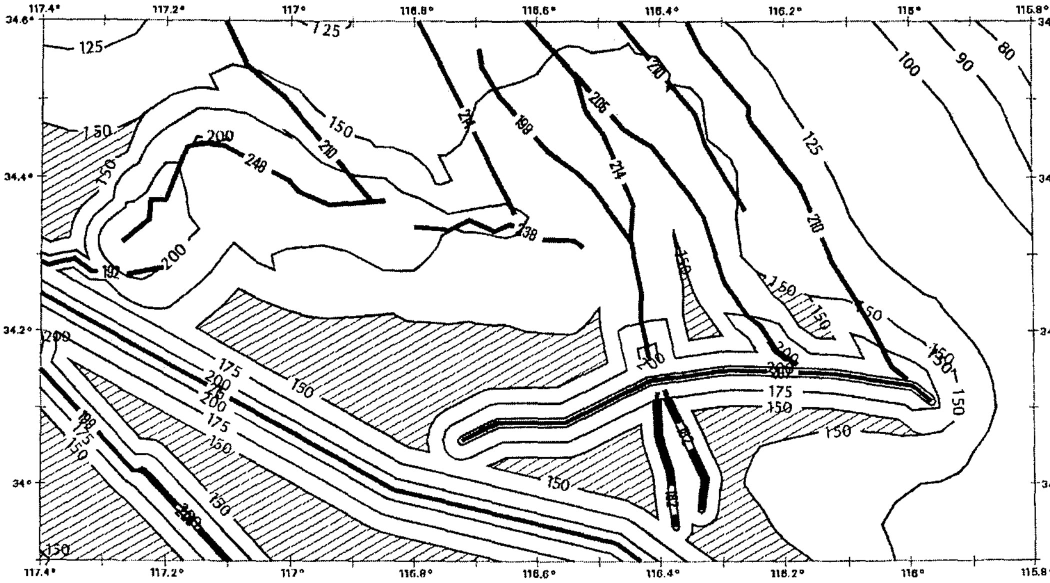


FIGURE B18c Combining contours - Example 1. Probabilistic contours are merged with deterministic contours using strict interpretation of committee rules with subsequent editing.

Another problem created was that some of the faults have portions of the fault, with a specific acceleration value, in areas where the contours are less than the fault value. An example occurs with the fault labeled 248 in the vicinity of 34.4° and 117.2° A footnote was added to the *maximum considered earthquake* maps to the effect that the fault value was only to be used in areas where it exceeded the surrounding contours. Although other approaches are possible, such as showing the unused portion of the fault dashed, the full length of the faults are shown solid in the maps.

As shown in Figure B18b, a sawtooth contour around 34.15° and 116.3° results from application of committee rules. Although this appears to be a candidate for smoothing, it was not done as shown in Figure B18c. Once again there are several possible ways to smooth but it was not done in the interest of repeatability.

Probability Level

The maximum considered earthquake spectral acceleration maps use the 2 percent in 50 maps as a base; however, the values obtained from the maps are multiplied by 2/3 for use in the design equation. This implicitly results in a different probability being used in different areas of the United States. The hazard curves shown in Figure B2 are normalized to the 2 percent in 50 year value in Figure B19. This figure shows that the slope of the hazard curve varies in different areas of the United States. In general, the curves are steeper for CEUS cities than for WUS cities with the WUS curves beginning to flatten out earlier than the CEUS cities. Typical curves for a CEUS and WUS city are shown in Figure B20. This figure shows than when the 2/3 factor is applied, probabilistic values a for WUS location are close to a 10 percent in 50 year value and probabilities for CEUS locations reflect a lower probability.

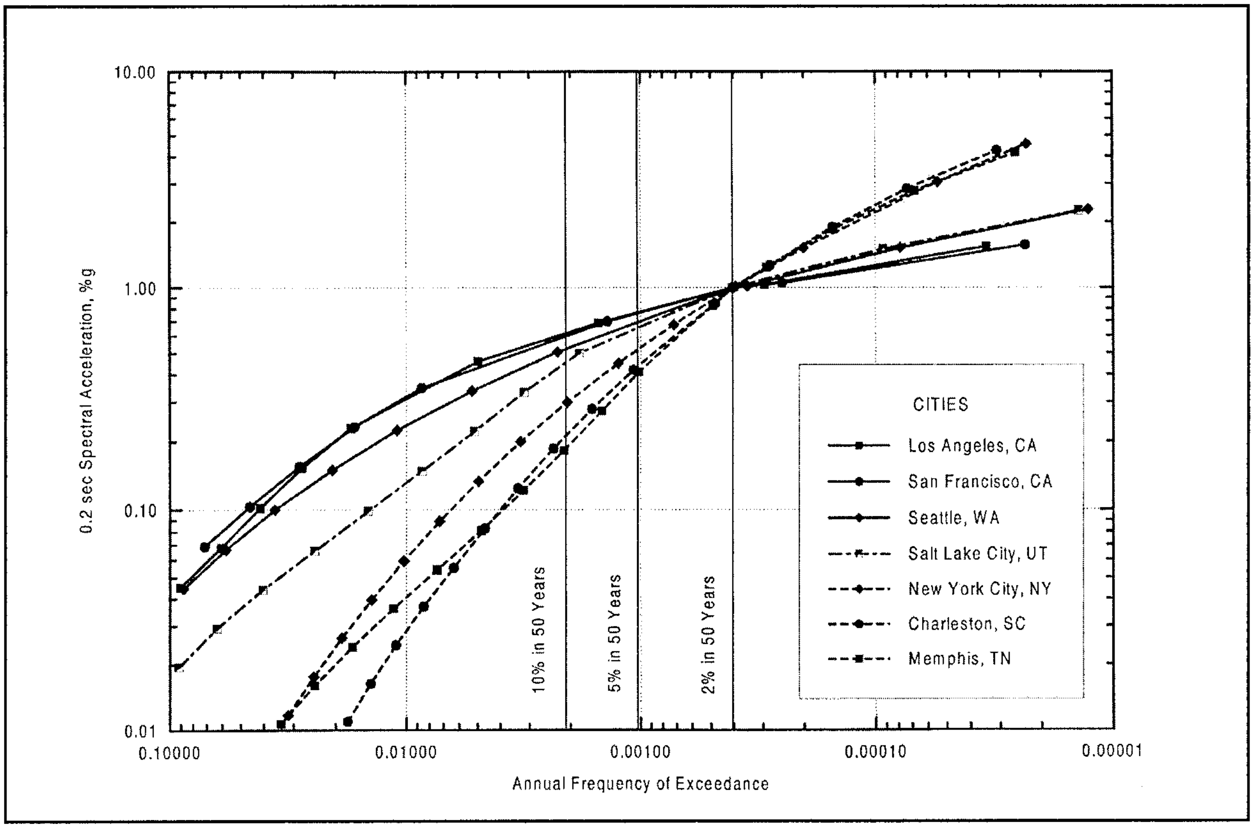


FIGURE B19 Hazard curves for selected cities. The curves are normalized to 2% in 50 years.

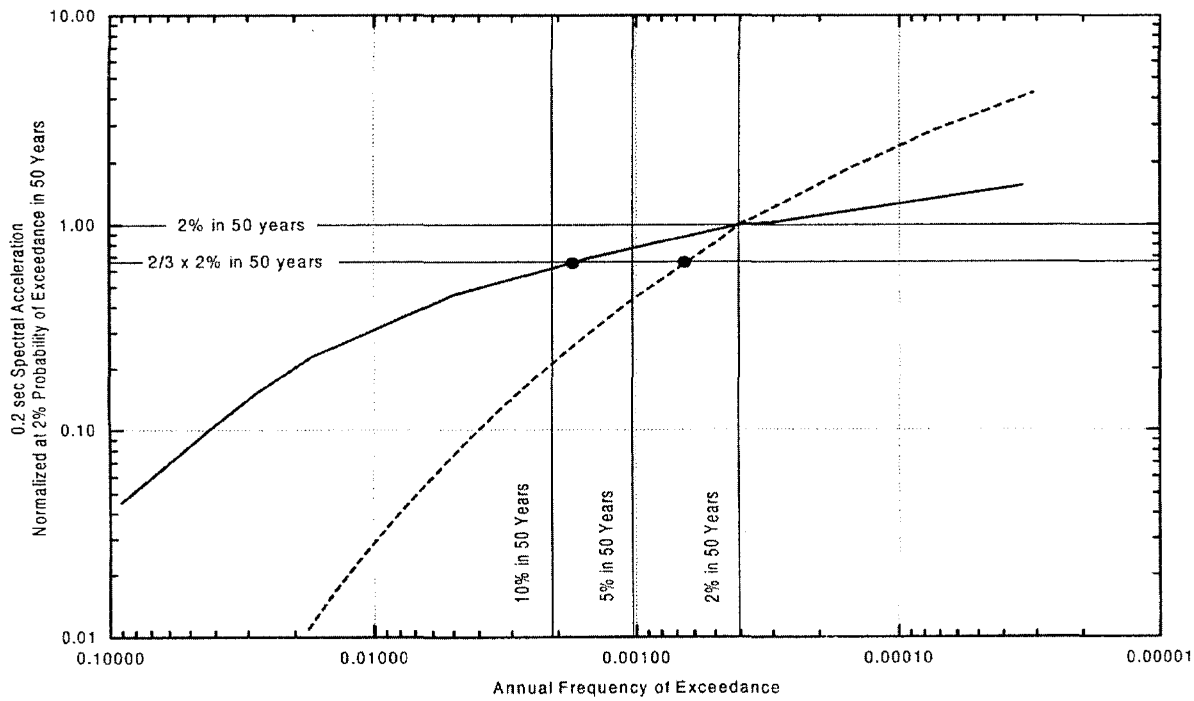


FIGURE B20 Effect on the probability level of multiplying the spectral acceleration by 2/3.

Interpolation

Linear interpolation between contours is permitted using the *maximum considered earthquake* maps. To facilitate interpolation, spot values have been provided inside closed contours of increasing or decreasing values of the design parameter. Additional spot values have been provided where linear interpolation would be difficult. Values have also been provided along faults in the deterministic areas to aid in interpolation.

Hawaii

The Hawaii State Earthquake Advisory Board (HSEAB), in its ballot on the 1997 *Provisions*, proposed different maps from those included in the original BSSC ballot. The HSEAB's comments were based in part on recent work done to propose changes in seismic zonation for the 1994 and 1997 *Uniform Building Code.* The HSEAB also was concerned that in early 1998 the USGS would be completing maps that would be more up to date then those included in the original BSSC ballot. Essentially, the HSEAB's recommendation was that the maps it submitted or the new USGS maps should be used for Hawaii. The USGS maps were completed in March 1998 and were reviewed by the HSEAB, including proposals for incorporation of deterministic contours where the ground motions exceed the plateau levels described previously. The maps were revised in response to review comments and the modified design maps are included as part of the *Provisions*.

Briefly, the probabilistic maps were prepared using a USGS methodology similar to that used for the western United States. Two attenuation fuctions were used: Sadigh as described earlier and Munson and Thurber, which incorportes Hawaii data. The Hawaii contour maps (*Provisions* Maps 19 and 20) are probabilistic except for two areas on the island of Hawaii. The two areas (outlined by the heavy border on Maps 19 and 20) are located on the western and southeastern portion of the island. The two areas are defined by horizontal rupture planes at a 9 km depth. Within these zones, the spectral accelerations are constant. The western zone uses a magniture 7.0 event while the southwestern zones uses a magnitude 8.2 event. The deterministic values inside the zone and for the contours were calculated as described in earlier sections.

Documentation for the maps is being prepared. The probabilistic maps and documentation are available on the USGS internet site (htt\p://geohazards.cr.usgs.gov/eq/)

Additional Maximum Considered Earthquake Ground Motion Maps

Although new probabilistic maps were not available for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, St. Croix, Guam, and Tutuila *maximum considered earthquake* maps were required for use by the *Provisions*. *Maximum considered earthquake* spectral response maps for these areas were prepared as follows.

Maps for Puerto Rico, Culebra, Vieques, St. Thomas, St. John, St. Croix, Guam, and Tutuila, were prepared using the 1994 NEHRP maps. These were for approximately 10 percent probability of exceedance in 50 years. The ratio of PGA for 2 percent in 50 years to 10 percent in 50 years for the new USGS maps is about two. Accordingly maps for these areas were converted to 2 percent in 50 year maps by multiplying by two. These maps were then converted to spectral maps by using the factors described below.

A study of the ratios of the 0.2 sec and 1.0 sec spectral responses to PGA was done. Although approximate, the ratios were about 2.25 to 2.5 for the 0.2 sec spectral acceleration and about 1.0 for the 1.0 sec response. Thus PGA for the above regions was converted to spectral acceleration by multiplying PGA by 2.5 for the 0.2 sec response and by 1.0 for the 1.0 sec response. It should be noted that the multiplier for the 1.0 sec response varied over a wider range than the 0.2 sec response multiplier. It should be used cautiously.

REFERENCES

Adams, J., D. H. Weichert, S. Halchuk, and P. W. Basham. 1996. Trial seismic hazard maps of Canada-1995: final values for selected Canadian cities, Geological Survey of Canada, Open File 3283.

Algermissen, S. T., D. M. Perkins, P. C. Thenhaus, S. L. Hanson, and B. L. Bender. 1990. *Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico*, U.S. Geological Survey, Miscelaneous Field Studies Map MF-2120.

Algermissen, S. T., E. V. Leyendecker, G. A. Bollinger, N. C. Donovan, J. E. Ebel, W. B. Joyner, R. W. Luft, and J. P. Singh. 1991. "Probabilistic Ground-motion Hazard Maps of Response

Spectral Ordinates for the United States," in *Proceedings of Fourth International Conference on Seismic Ordinates for the United States*, Stanford Univ., v. II, pp. 687-694.

Amick , D. and R. Gelinas. 1991. "The Search for Evidence of Large Prehistoric Earthquakes along the Atlantic Seaboard," *Science,* v. 251, pp. 655-658.

Atwater, B .F., and E. Hemphill-Haley . 1996. Preliminary estimates of recurrence intervals for great earthquakes of the past 3500 years at northeastern Willapa Bay, Washington, U.S. Geological Survey, Open-file report 96-001, 88pp.

Atwater, B., A. Nelson, J. Clague, G. Carver, D. Yamaguchi, P. Bobrowsky, J. Bourgeois, M. Darienzo, W. Grant, S. Palmer, C. Peterson, and M. Reinhart . 1995. Summary of coastal geologic evidence for past great earthquakes at the Cascadia subduction zone, Earthquake Spectra, v. 11, pp. 1-18.

Black, B. D., W. R. Lund, and B. H. Mayes . 1995. Large earthquakes on the Salt Lake City segment of the Wasatch Fault Zone--summary of new information from the South Fork Dry Creek site, Salt Lake County, Utah, in Environmental and Engineering Geology of the Wasatch Front Region, W. R. Lund, ed., Utah Geological Association Publication 24, 1995.

Boore, D. M., W. B. Joyner, and T. E. Fumal . 1993. Estimation of response spectra and peak accelerations from western North American earthquakes: an interim report, U.S. Geological Survey open-file report 93-509.

Boore, D. M., W. B. Joyner, and T. E. Fumal . 1994a. Estimation of response spectra and peak accelerations from western North American earthquakes: an interim report Part 2, U.S. Geological Survey open-file report 94-127.

Boore, D. M., W. B. Joyner and T. E. Fumal. 1994b. Ground motion estimates for strike- and reverse-slip faults . pre-print.

Boore, D. M., W. B. Joyner, and T. E. Fumal . 1997. Equations for estimating horizontal response spectra and peak acceleration from western North American earthquakes: a summary of recent work, Seismological Research Letters, v.68, no. 1, pp. 128-153.

Chandler, V. W. 1995. The west-central Minnesota earthquake of June 5, 1993: an opportunity to re-examine seismicity near the Morris Fault, Seism. Res. Letts, v. 66, no. 6, pp. 113-121.

Crone, A. J., and K. V. Luza. 1990. "Style and Timing of Holocene Surface Faulting on the Meers Fault, Southwestern Oklahoma," *Geological Societ of America Bull*etin, v. 102, pp. 1-17.

Crone, A., M. Machette, L. Bradley and S. Mahan. 1996. *Holocene Surface Faulting on the Cheraw Fault, Southeastern Colorado,* U.S. Geological Survey, MF map, in press.

Crouse, C. B. 1991. Ground-motion attenuation equations for earthquakes on the Cascadia subduction zone, Earthquake Spectra, v. 7, p. 201.

Doser, D. I. 1985. Source parameters and faulting processes of the 1959 Hebgen Lake, Montana, earthquake sequence, J. Geophys. Res., v. 90, pp. 4537-4556.

Electric Power Research Institute. 1986. Seismic hazard methodology for the central and eastern United States, 10 volumes, EPRI report NP-4726, Electric Power Research Institute, Palo Alto.

Edge, K. M., B. A. Scheol, and I. Po Lam. 1992. Development of seismic maps for Arizona, Arizona Dept. of Transportation, report no. FHWA-AZ92-344.

Frankel, A. 1995. "Mapping seismic hazard in the Central and Eastern United States", *Seismological Research Letters*, v. 66, no. 4, pp. 8-21.

Frankel A., C. Mueller, T. Barnhard, D. Perkins, E. Leyendecker, N. Dickman, S. Hanson, and M. Hopper. 1996. *National Seismic Hazard Maps, June 1996: Documentation*, U.S. Geological Survey, Open-file Report 96-532.

Frankel A., C. Mueller, T. Barnhard, D. Perkins, E. Leyendecker, N. Dickman, S. Hanson, and M. Hopper. 1997a. *Seismic - Hazard Maps for the Conterminous United States:* U.S. Geological Survey, Open-file Report 97-130, 12 sheets, scale 1:7,000,000.

Frankel A., C. Mueller, T. Barnhard, D. Perkins, E. Leyendecker, N. Dickman, S. Hanson, and M. Hopper. 1997b. *Seismic - Hazard Maps for the California, Nevada and Western Arizona/Utah:* U.S. Geological Survey, Open-file Report 97-130, 12 sheets, scale 1:2,000,000.

Geomatrix Consultants. 1993. Seismic margin earthquake for the Trojan site: final unpublished report prepared for Portland General Electric Trojan Nuclear Plant, Ranier, Oregon. Note: tables of these attenuation relationships are included in Geomatrix , 1995.

Geomatrix Consultants. 1995. Seismic design mapping state of Oregon, Final Report prepared for Oregon Department of Transportation, Salem, Oregon.

Goldfinger, C., L. D. Kulm, R. S. Yeats, B. Applegate, M. MacKay, and G. R.Cochrane . in press. Active strike-slip faulting and folding of the Cascadia plate boundary and forearc in central and northern Oregon, in Rogers, A. M., W. J. Kockelman, G. Priest, and T. J. Walsh . eds., Assessing and reducing earthquake hazards in the Pacific northwest, U.S. Geological Survey Professional Paper 1560.

Hyndman, R. D., and K. Wang. 1995. The rupture zone of Cascadia great earthquakes from current deformation and the thermal regime, *J. Geophys. Res.*, v. 100, pp. 22,133-22,154.

Johnston, A. 1994. Moment magnitude assessment of stable continental earthquakes, Part I: instrumental seismicity, pre-print . published with revised relations in Geophys. J. Int., [1996], v. 124, pp. 381-414

Johnston, A. 1996a. "Moment Magnitude Assessment of Stable Continental Earthquakes, Part 2: Historical Seismicity," *Geophys. J. Int.*, v. 125, pp. 639-678.

Johnston, A. 1996b. "Seismic Moment Assessment of Stable Continental Earthquakes, Part 3: 1811-1812 New Madrid, 1886 Charleston, and 1755 Lisbon," submitted to *Geophys. J. Int.*

Johnston, A. C., K. J. Coppersmith, L. R. Kanter, and C. A. Cornell. 1994. The earthquakes of stable continental regions: assessment of large earthquake potential, EPRI TR-102261, J. F. Schneider, ed., Electric Power Research Institute, 309 pp.

Johnston, A. C., and E. S. Schweig . 1996. The enigma of the New Madrid earthquakes of 1811-1812, Annual Review of Earth and Planetary Sciences, v. 24, pp. 339-384.

Joyner, W. B. 1995. Advocacy Paper #4, Soil is the most appropriate reference site condition, in ATC 35-2, Preprints: National Earthquake Ground Motion Mapping Workshop, Applied Technology Council.

Leyendecker, E. V., D. M. Perkins, S. T. Algermissen, P. C. Thenhaus, and S. L. Hanson. 1995. *USGS Spectral Response Maps and Their Relationship with Seismic Design Forces in Building Codes*, U.S. Geological Survey, Open-File Report 95-596.

Lienkaemper, J. J. 1996. "Fault parameter compilation for northern California", in preparation.

Martin, G. R., and R. Dobry. 1994. Earthquake site response and seismic code provisions, NCEER Bulletin, v. 8, pp. 1-6.

Mueller, C., M. Hopper, and A. Frankel. 1996. Preparation of earthquake catalogs for the 1996 national seismic hazard maps: documentation, U.S. Geological Survey Open-File Report, in review.

Mueller, C., M. A. Frankel, J. Lahr, and M. Wyss. 1997. Preparation of earthquake catalogs for the National Seismic Hazard Maps: Alaska, U.S. Geological Survey Open-File Report, in review.

Obermeier, S. F., R. B. Jacobson, J. P. Smoote, R. E. Weems, G. S. Gohn. J. E. Monroe, and D. S. Powers. 1990. "Earthquake-induced Liquefaction Features in the Coastal Setting of South Carolina and in the Fluvial Setting of the New Madrid Seismic Zone," U.S. Geological Survey, Prof. Paper 1504, 44 pp.

Obermeier, S. F., P. J. Munson, C. A. Munson, J. R. Martin, A .D. Frankel, T. L. Youd, and E. C. Pond. 1992. "Liquefaction Evidence for Strong Holocene Earthquake. s in the Wabash Valley of Indiana-Illinois," *Seismological Research Letters,* v. 63, pp. 321-336.

Petersen, M. D., and S. G. Wesnousky. 1994. Fault slip rates and earthquake histories for active faults in southern California, *Bull. Seism. Soc. Am.*, v. 84, pp. 1608-1649.

Petersen, M. D., C. H. Cramer, W. A. Bryant, M. S. Reichle, and T. R. Toppozada. 1996. Preliminary hazard assessment for Los Angeles, Ventura, and Orange counties, California, affected by the 17 January 1994 Northridge earthquake, *Bull. Seism. Soc. Am.*, v. 86, pp. S247-S261.

Powell, C., G. Bollinger, M. Chapman, M. Sibol, A. Johnston, and R. Wheeler. 1994. A seismotectonic model for the 300-kilometer-long eastern Tennessee seismic zone, *Science*, v. 264, pp. 686-688.

Sadigh, K., C. Y. Chang, N. A.Abrahamson, S. J. Chiou, and M. S. Power. 1993. Specification of long-period ground motions: updated attenuation relationships for rock site conditions and adjustment factors for near-fault effects, in Proceedings of ATC-17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control, Applied Technology Council, Redwood City. Note: tables of these attenuation relationships are included in Geomatrix, 1995.

Sadigh, K., C. Y. Chang, J. Egan, F. Makdisi, and R. Youngs. 1997. Attenuation relationships for shallow crustal earthquakes based on California strong motion data, *Seismological Research Letters*, v.68, no. 1, pp. 180-189.

Satake, K., K. Shimazaki, Y. Tsuji, and K. Ueda. 1996. Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami record of January 1700, Nature, v. 379, pp. 246-249.

Schwartz, D. P., and K. J. Coppersmith. 1984. Fault behavior and characteristic earthquakes: examples from the Wasatch and San Andreas fault zones, J. Geophys. Res., v. 89, pp. 5681-5698.

Seeber, L., and J. G. Armbruster. 1991. *The NCEER-91 Earthquake Catalog: Improved Intensity-based Magnitudes and Recurrence Relations for U.S. Earthquakes East of New Madrid*, National Center for Earthquake Engineering Research, NCEER-91-0021.

Senior Seismic Hazard Analysis Committee . 1996?. *Probabilistic Seismic Hazard Analysis: a Consensus Methodology,* supported by DOE, NRC, and EPRI.

Sibol, M. S., G. A. Bollinger, and J. B. Birch. 1987. "Estimation of Magnitudes in Central and Eastern North America Using Intensity and Felt Area," *Bulletin of the Seismological Society of America,* v. 77, pp. 1635-1654.

Stover, C. W., and J. L. Coffman. 1993. " Seismicity of the United States 1568-1989 . revised," *U.S. Geological Survey Professional Paper 1527.*

Stover, C. W., G. Reagor, and S. T. Algermissen. 1984. *United States Earthquake Data File,* U.S. Geological Survey Open-File Report 84-225.

Toro, G. R., N. A. Abrahamson, and J. F. Schneider. 1993. "Engineering Model of Strong Ground Motions from Earthquakes in the Central and Eastern United States," in *Guidelines for Determining Design Basis Ground Motions*, EPRI TR-102293, J. F. Schneider, ed., Electric Power Research Institute.

U.S. Geological Survey National Seismic-Hazard Mapping Project, 1998a, *Alaska Seismic-Hazard Maps: Documentation*, U.S. Geological Survey, Open-file Report, in progress.

U.S. Geological Survey National Seismic-Hazard Mapping Project, 1998b, *Seismic-Hazard Maps of Alaska:* U.S. Geological Survey, Open-file Report, 6 sheets, scale 1:5,000,000, in progress.

Ward, S. N. 1994. A multidisciplinary approach to seismic hazard in southern California, Bull. Seism. Soc. Am., v.84, pp. 1293-1309.

Weichert, D. H. 1980. Estimation of earthquake recurrence parameters for unequal observation periods for different magnitudes, Bull. Seism. Soc. Am., 70, pp. 1337-1356.

Wells, D. L., and K. J. Coppersmith. 1994. "New empirical relationships among magnitude, rupture length, rupture width, and surface displacement," *Bulletin of the Seismological Society of America,* v. 84, pp. 974-1002.

Wesnousky, S. G. 1986. Earthquakes, Quaternary faults, and seismic hazard in California, J. Geophys. Res., v. 91, pp. 12,587-12,631.

Wheeler, R. L. 1995. "Earthquakes and the Cratonward Limit of Iapetan Faulting in Eastern North America," *Geology*, v. 23, pp. 105-108.

Working Group on California Earthquake Probabilities. 1995. Seismic hazards in southern California: probable earthquakes, 1994 to 2024, Bull. Seism. Soc. Am., v. 85, pp. 379-439.

Wyman, R.V., M. Karakouzian, V. Bax-Valentine, D. B. Slemmons, L. Peterson, and S. Palmer . 1993. Geology of Las Vegas, Nevada, Bull. Assoc. of Engineer. Geologists, v. XXX, pp. 33-78.

Youngs, R., Chiou, W. Silva, and J. Humphrey. 1997. Strong ground motion attenuation relationships for subduction zone earthquakes, Seismological Research Letters, v.68, no. 1, pp. 180-189.

BUILDING SEISMIC SAFETY COUNCIL

Of the National Institute of Building Sciences

THE COUNCIL: ITS PURPOSE AND ACTIVITIES

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

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. To fulfill its purpose, the BSSC:

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

The BSSC's area of interest encompasses all building types, structures, and related facilities and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. The BSSC believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are structured to provide all interested entities (i.e., government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community. The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard risk reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. Thus, the BSSC itself assumes no standards-making or -promulgating role; rather, it advocates that code- and standards-formulation organizations consider the BSSC's recommendations for inclusion in their documents and standards.

IMPROVING THE SEISMIC SAFETY OF NEW BUILDINGS

The BSSC program directed toward improving the seismic safety of new buildings has been conducted with funding from the Federal Emergency Management Agency (FEMA). It is structured to create and maintain authoritative, technically sound, up-to-date resource documents that can be used by the voluntary standards and model code organizations, the building community, the research community, and the public as the foundation for improved seismic safety design provisions.

The BSSC program began with initiatives taken by the National Science Foundation (NSF). Under an agreement with the National Institute of Standards and Technology (NIST; formerly the National Bureau of Standards), *Tentative Provisions for the Development of Seismic Regulations for Buildings* (referred to here as the *Tentative Provisions*) was prepared by the Applied Technology Council (ATC). The ATC document was described as the product of a "cooperative effort with the design professions, building code interests, and the research community" intended to "...present, in one comprehensive document, the current state of knowledge in the fields of engineering seismology and engineering practice as it pertains to seismic design and construction of buildings." The document, however, included many innovations, and the ATC explained that a careful assessment was needed.

Following the issuance of the *Tentative Provisions* in 1978, NIST released a technical note calling for ". . . systematic analysis of the logic and internal consistency of [the *Tentative Provisions*]" and developed a plan for assessing and implementing seismic design provisions for buildings. This plan called for a thorough review of the *Tentative Provisions* by all interested organizations; the conduct of trial designs to establish the technical validity of the new provisions and to assess their economic impact; the establishment of a mechanism to encourage consideration and adoption of the new provisions by organizations promulgating national standards and model codes; and educational, technical, and administrative assistance to facilitate implementation and enforcement.

During this same period, other significant events occurred. In October 1977, Congress passed the *Earthquake Hazards Reduction Act of 1977* (P.L. 95-124) and, in June 1978, the National Earthquake Hazards Reduction Program (NEHRP) was created. Further, FEMA was established as an independent agency to coordinate all emergency management functions at the federal level. Thus, the future disposition of the *Tentative Provisions* and the 1978 NIST plan shifted to FEMA. The emergence of FEMA as the agency responsible for implementation of P.L. 95-124 (as amended) and the NEHRP also required the creation of a mechanism for obtaining broad public and private consensus on both recommended improved building design and construction regulatory provisions and the means to be used in their promulgation. Following a series of meetings between representatives of the original participants in the NSF-sponsored project on seismic design provisions, FEMA, the American Society of Civil Engineers and the National Institute of Building Sciences (NIBS), the concept of the Building Seismic Safety Council was born. As the concept began to take form, progressively wider public and private participation was sought, culminating in a broadly representative organizing meeting in the spring of 1979, at which time a charter and organizational rules and procedures were thoroughly debated and agreed upon.

The BSSC provided the mechanism or forum needed to encourage consideration and adoption of the new provisions by the relevant organizations. A joint BSSC-NIST committee was formed to conduct the needed review of the *Tentative Provisions*, which resulted in 198 recommendations for changes. Another joint BSSC-NIST committee developed both the criteria by which the needed trial designs could be evaluated and the specific trial design program plan. Subsequently, a BSSC--NIST Trial Design Overview Committee was created to revise the trial design plan to accommodate a multiphased effort and to refine the *Tentative Provisions*, to the extent practicable, to reflect the recommendations generated during the earlier review.

Trial Designs

Initially, the BSSC trial design effort was to be conducted in two phases and was to include trial designs for 100 new buildings in 11 major cities, but financial limitations required that the program be scaled down. Ultimately, 17 design firms were retained to prepare trial designs for 46 new buildings in 4 cities with medium to high seismic risk (10 in Los Angeles, 4 in Seattle, 6 in Memphis, 6 in Phoenix) and in 5 cities with medium to low seismic risk (3 in Charleston, South Carolina, 4 in Chicago, 3 in Ft. Worth, 7 in New York, and 3 in St. Louis). Alternative designs for six of these buildings also were included.

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

For each of the 52 designs, a set of general specifications was developed, but the responsible design engineering firms were given latitude to ensure that building design parameters were compatible with local construction practice. The designers were not permitted, however, to change the basic structural type even if an alternative structural type would have cost less than the specified type under the early version of the *Provisions*, and this constraint may have prevented some designers from selecting the most economical system.

Each building was designed twice – once according to the amended *Tentative Provisions* and again according to the prevailing local code for the particular location of the design. In this context, basic structural designs (complete enough to assess the cost of the structural portion of the building), partial structural designs (special studies to test specific parameters, provisions, or objectives), partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building), and design/construction cost estimates were developed.

This phase of the BSSC program concluded with publication of a draft version of the recommended provisions, the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, an overview of the *Provisions* refinement and trial design efforts, and the design firms' reports.

The 1985 Edition of the *NEHRP Recommended Provisions*

The draft version represented an interim set of provisions pending their balloting by the BSSC member organizations. The first ballot, conducted in accordance with the BSSC Charter, was organized on a chapter-by-chapter basis. As required by BSSC procedures, the ballot provided for four responses: "yes," "yes with reservations," "no," and "abstain." All "yes with reservations" and "no" votes were to be accompanied by an explanation of the reasons for the vote and the "no" votes were to be accompanied by specific suggestions for change if those changes would change the negative vote to an affirmative.

All comments and explanations received with "yes with reservations" and "no" votes were compiled, and proposals for dealing with them were developed for consideration by the Technical Overview Committee and, subsequently, the BSSC Board of Direction. The draft provisions then were revised to reflect the changes deemed appropriate by the BSSC Board and the revision was submitted to the BSSC membership for balloting again.

As a result of this second ballot, virtually the entire provisions document received consensus approval, and a special BSSC Council meeting was held in November 1985 to resolve as many of the remaining issues as possible. The 1985 Edition of the *NEHRP Recommended Provisions* then was transmitted to FEMA for publication in December 1985.

During the next three years, a number of documents were published to support and complement the 1985 *Provisions*. They included a guide to application of the *Provisions* in earthquake-resistant building design, a nontechnical explanation of the *Provisions* for the lay reader, and a handbook for interested members of the building community and others explaining the societal implications of utilizing improved seismic safety provisions and a companion volume of selected readings.

The 1988 Edition

The need for continuing revision of the *Provisions* had been anticipated since the onset of the BSSC program and the effort to update the 1985 Edition for reissuance in 1988 began in January 1986. During the update effort, nine BSSC Technical Committees (TCs) studied issues concerning seismic risk maps, structural design, foundations, concrete, masonry, steel, wood, architectural and mechanical and electrical systems, and regulatory use. The Technical Committees worked under the general direction of a Technical Management Committee (TMC), which was composed of a representative of each TC as well as additional members identified by the BSSC Board to provide balance.

The TCs and TMC worked throughout 1987 to develop specific proposals for changes needed in the 1985 *Provisions*. In December 1987, the Board reviewed these proposals and decided upon a set of 53 for submittal to the BSSC membership for ballot. Approximately half of the proposals reflected new issues while the other half reflected efforts to deal with unresolved 1985 edition issues.

The balloting was conducted on a proposal-by-proposal basis in February-April 1988. Fifty of the proposals on the ballot passed and three failed. All comments and "yes with reservation" and "no" votes received as a result of the ballot were compiled for review by the TMC. Many of the comments could be addressed by making minor editorial adjustments and these were approved by the BSSC Board. Other comments were found to be unpersuasive or in need of further study during the next update cycle (to prepare the 1991 *Provisions*). A number of comments persuaded the TMC and Board that a substantial alteration of some balloted proposals was necessary, and it was decided to submit these matters (11 in all) to the BSSC membership for reballot during June-July 1988. Nine of the eleven reballot proposals passed and two failed.

On the basis of the ballot and reballot results, the 1988 *Provisions* documents were prepared and transmitted to FEMA for publication in August 1988. A report describing the changes made in the 1985 edition and issues in need of attention in the next update cycle also was prepared, and efforts to update the complementary reports published to support the 1985 edition were initiated. Ultimately, the following publications were updated to reflect the 1988 Edition and reissued by FEMA: the *Guide to Application of the Provisions*, the handbook discussing societal implications (which was extensively revised and retitled *Seismic Considerations for Communities at Risk*), and several *Seismic Considerations* handbooks (which are described below).

The 1991 Edition

During the effort to produce the 1991 *Provisions*, a Provisions Update Committee (PUC) and 11 Technical Subcommittees addressed seismic hazard maps, structural design criteria and analysis, foundations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite structures. Their work resulted in 58 substantive and 45 editorial proposals for change to the 1988 *Provisions*.

The PUC approved more than 90 percent of the proposals and, in January 1991, the BSSC Board accepted the PUC-approved proposals for balloting by the BSSC member organizations in April-May 1991.

Following the balloting, the PUC considered the comments received with "yes with reservations" and "no" votes and prepared 21 reballot proposals for consideration by the BSSC member organizations. The reballoting was completed in August 1991 with the approval by the BSSC member organizations of 19 of the reballot proposals.

On the basis of the ballot and reballot results, the 1991 *Provisions* documents were prepared and transmitted to FEMA for publication in September 1991. Reports describing the changes made in the 1988 Edition and issues in need of attention in the next update cycle also were developed.

In August 1992, in response to a request from FEMA, the BSSC initiated an effort to continue its structured information dissemination and instruction/training effort aimed at stimulating widespread use of the *Provisions*. The primary objectives of the effort were to bring several of the publications complementing the *Provisions* into conformance with the 1991 Edition in a manner reflecting other related developments (e.g., the fact that all three model codes now include requirements based on the *Provisions*) and to bring instructional course materials currently being used in the BSSC seminar series (described below) into conformance with the 1991 *Provisions*.

The 1994 Edition

The effort to structure the 1994 PUC and its technical subcommittees was initiated in late 1991. By early 1992, 12 Technical Subcommittees (TSs) were established to address seismic hazard mapping, loads and analysis criteria, foundations and geotechnical considerations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite steel and concrete structures, and base isolation/energy dissipation.

The TSs worked throughout 1992 and 1993 and, at a December 1994 meeting, the PUC voted to forward 52 proposals to the BSSC Board with its recommendation that they be submitted to the BSSC member organizations for balloting. Three proposals not approved by the PUC also were forwarded to the Board because 20 percent of the PUC members present at the meeting voted to do so. Subsequently, an additional proposal to address needed terminology changes also was developed and forwarded to the Board.

The Board subsequently accepted the PUC-approved proposals; it also accepted one of the proposals submitted under the "20 percent" rule but revised the proposal to be balloted as four separate items. The BSSC member organization balloting of the resulting 57 proposals occurred in March-May 1994, with 42 of the 54 voting member organizations submitting their ballots. Fifty-three of the proposals passed, and the ballot results and comments were reviewed by the PUC in July 1994. Twenty substantive changes that would require reballoting were identified. Of the four proposals that failed the ballot, three were withdrawn by the TS chairmen and one was substantially modified and also was accepted for reballoting. The BSSC Board of Direction accepted the PUC recommendations except in one case where it deemed comments to be persuasive and made an additional substantive change to be reballoted by the BSSC member organizations.

The second ballot package composed of 22 changes was considered by the BSSC member organizations in September-October 1994. The PUC then assessed the second ballot results and made its recommendations to the BSSC Board in November. One needed revision identified later was considered by the PUC Executive Committee in December. The final copy of the 1994 Edition of the *Provisions* including a summary of the differences between the 1991 and 1994 Editions was delivered to FEMA in March 1995.

The 1997 Edition

In September 1994, NIBS entered into a contract with FEMA for initiation of the 39-month BSSC 1997 *Provisions* update effort. Late in 1994, the BSSC member organization representatives and alternate representatives and the BSSC Board of Direction were asked to identify individuals to serve on the 1997 PUC and its TSs. The 1997 PUC was constituted early in 1995, and 12 PUC Technical Subcommittees were established to address design criteria and analysis, foundations and geotechnical considerations, cast-in-place/precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, composite steel and concrete structures, energy dissipation and base isolation, and nonbuilding structures.

As part of this effort, the BSSC developed for the 1997 *Provisions* a revised seismic design procedure. Unlike the design procedure based on U.S. Geological Survey (USGS) peak acceleration and peak velocity-related acceleration ground motion maps developed in the 1970s and used in earlier editions of the *Provisions*, the new design procedure involves new design maps based on recently revised USGS spectral response maps and a process specified within the body of the *Provisions*. This task was conducted with the cooperation of the USGS (under a Memorandum of Understanding signed by the BSSC and USGS) by the Seismic Design Procedure Group (SDPG) working with the guidance of a five-member Management Committee.

More than 200 individuals participated in the 1997 update effort, and more than 165 substantive proposals for change were developed. A series of editorial/organizational changes also were made. All draft TS, SDPG, and PUC proposals for change were finalized in late February 1997, and in early March, the PUC Chair presented to the BSSC Board of Direction the PUC's recommendations concerning proposals for change to be submitted to the BSSC member organizations for balloting. The Board accepted these recommendations, and the first round of balloting was conducted in April-June 1997.

Of the 158 items on the first ballot, only 8 did not pass; however, many comments were submitted with "no" and "yes with reservations" votes. These comments were compiled for distribution to the PUC, which met in mid-July to review the comments, receive TS responses to the comments and recommendations for change, and formulate its recommendations concerning what items should be submitted to the BSSC member organizations for a second ballot. The PUC deliberations resulted in the decision to recommend to the BSSC Board that 28 items be included in the second ballot. The PUC Chair subsequently presented the PUC's recommendations to the Board, which accepted those recommendations.

The second round of balloting was completed in October. All but one proposal passed; however, a number of comments on virtually all the proposals were submitted with the ballots and were immediately compiled for consideration by the PUC. The PUC Executive Committee met in December to formulate its recommendations to the Board, and the Board subsequently accepted those recommendations.

The PUC concluded its update work by identifying issues in need of consideration during the next update cycle and technical issues in need of study. The final version of the 1997 *Provisions*, including an appendix describing the differences between the 1994 and 1997 edition, was transmitted to FEMA in February 1998. The contract for the 1997 update effort was extended by FEMA to September 1999 to permit several complementary initiatives to be pursued.

One of these initiatives resulted in a CD that provides all of the design mapping data needed for use with the 1997 *NEHRP Recommended Provisions* and *International Building Code* as well as the *International Residential Code* and the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. This CD was developed for the BSSC by Dr. E. V. Leyendecker of the U.S. Geological Survey and should be available very soon. It permits the user to search either by longitude and latitude or by zipcode. Delivery in early 2000 is expected. Although the CD-ROM will be distributed by FEMA and the BSSC, it is anticipated that the ICC will be permitted to reproduce copies to accompany the *IBC*.

The second initiative resulted in a list of the relevant seismic design map data on a county-by-county basis. One listing will identify populated places, state, county, population (when available), latitude and longitude, two maximum considered earthquake (MCE) spectral points (for use with the 1997 *NEHRP Recommended Provisions, International Building Code* and, to some extent, the *International Residential Code*); two spectral points for the 10 percent probability in 50 year maps (for use with the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*), and the corresponding category for use with the *International Residential Code*. The final version of this listing can be sorted alphabetically by county and then by place in the county. Another listing presents the counties for each state and provides the same information as in the first listing but uses the approximate geographic or "centroid" coordinates to determine the data grid values for each county as a whole. These listings are based on the CD and were developed for the BSSC by Richard McConnell.

In a somewhat related effort, the BSSC commissioned a set of approximately 40 comparative designs. Each comparative design was performed at least three times: once according to the proposed 2000 *IBC* (which is being take to represent the 1997 *NEHRP Recommended Provisions*), once according to the 1991 *Provisions* (requirements reflected in the *National Building Code* and *Standard Building Code*), and once according to the 1994 *Uniform Building Code*. Performing the study for the BSSC were the J. R. Harris and Company and S. K. Ghosh Associates, Inc. Copies of a summary of the study are expected to be available in spring 2000.

The new BSSC Internet web site – www.bssconline.org – is up and running. It permits visitors to search and/or download the 1997 *Provisions* and *Commentary*, write for technical assistance from *Provisions* experts, and review frequently asked questions. In addition, the site features password-protected areas where the PUC and its Technical Subcommittees post and discuss draft proposals for change. The proposals submitted to the BSSC member organizations in March 2000 were posted on the site and it is anticipated that second ballot proposals also will be posted (in September 2000).

2000 Edition

In September 1997, NIBS entered into a contract with FEMA for initiation of the 48-month BSSC effort to update the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. During this project, the 1997 *Provisions* is being revised for reissuance in 2000 and code changes based on the 2000 *Provisions* are being prepared for submittal to the *IBC*.

In lieu of the Seismic Design Procedure Group (SDPG) used in the 1997 update, the BSSC has re-established Technical Subcommittee 1, Seismic Design Mapping, used in earlier updates of the *Provisions*. This subcommittee is composed of an equal number of representatives from the earth science community, including representatives from the USGS, and the engineering community.

An additional 11 subcommittees were formed to address seismic design and analysis, foundations and geotechnical considerations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, composite steel and concrete structures, base isolation and energy dissipation, and nonbuilding structures. Two ad hoc task groups also were formed: one to develop appropriate anchorage requirements for concrete/masonry/wood elements and the other to develop a simplified procedure for use in the lower seismic risk areas. No technical subcommittee was established in this update cycle to serve specifically as the interface with codes and standards; rather, the Code Resource Support Committee and its Technical Group (see Section 4.2.2) provided for the needed liaison between the PUC and the model code and standards organizations.

The BSSC, through the PUC and its TSs, identified major technical issues to be addressed during the 2000 update of the *NEHRP Recommended Provisions*, assessed the basis for change to the 1997 Edition, resolve technical issues, and developed proposals for change. The results of recent relevant research and lessons learned from earthquakes occurring prior to or during the duration of the project have been given consideration at all stages of this process. Particular attention was on those technical problems identified but unresolved during the preparation of the 1997 Edition. The PUC also has coordinated its efforts with those individuals working with the ICC to develop the *IBC*. Changes recommended by those individuals were submitted to the PUC for consideration and changes developed by the PUC are being formatted for consideration in the *IBC* development process. As part of the update process, the BSSC also has worked to develop a simplified design procedure in order to improve use of the *Provisions* in areas of low and moderate seismic hazard.

The first ballot encompassing 146 proposals for change to the 1997 *Provisions was* submitted to the BSSC member organizations on April 10, 2000; the ballot deadline was June 10. The proposals for change also were posted for comment on the BSSC website – www.bssconline.org. Of the 64 member organizations sent ballot packages, 42 responded. Of the 146 proposals, 69 passed with no "no" votes but some "yes with reservations" votes, 71 passed but with "no" and "yes with reservations" votes, and 6 did not pass (i.e., received less than 67 percent "yes" and "yes with reservations" votes). The comments submitted with "no" and "yes with reservations" votes were compiled and distributed to the PUC Technical Subcommittee chairs. The PUC then met in Denver on July 13-14, 2000, to receive the TSs responses to ballot comments and formulate recommendations concerning items that need to be submitted to the member organizations for a second ballot

In August 2000, the PUC Chair, William Holmes, briefed the BSSC Board of Direction on the results of the first ballot and recommended that 17 items be submitted to the membership for a second ballot. Ten of the proposals were revisions of previous proposals, three were new proposals, and four were proposals developed by the PUC to clarify concerns arising from the first ballot. The official second ballot package was mailed to BSSC member organizations on September 5, 2000 with completed ballots due October 5, 2000. Of the 66 BSSC member organizations, 42 responded and all proposals passed. There were, however, several "yes with reservations" and "no" votes, and the PUC met on October 30-31, 2000, to resolve the comments submitted with these votes and to formulate recommendations concerning a third ballot.

On November 1, 2000, the PUC chair presented the second ballot results to the BSSC Board and recommended that several items be submitted to the membership for a third ballot. The primary purpose of the third ballot was to permit integration into the 2000 *Provisions* of new steel requirements resulting from the FEMA-funded SAC effort mounted to study damage during the Northridge earthquake and of the most current version of the American Institute of Steel Constructions standard which was expected to include many of the SAC requirements. The third ballot, which included five proposals, was sent to the membership on December 28, 2000, with ballot due by February 7, 2001. Of the 65 member organizations, 44 submitted ballots (67 percent). All five proposals passed and the results were reviewed and comments resolved by the PUC Executive Committee at a meeting on March 5, 2001.

The PUC chair briefed the BSSC Board on the third ballot results on March 6, 2000, and the Board unanimously approved the 2000 *Provisions* for transmittal to FEMA following a final editorial review by the PUC of the *Provisions* document and its accompanying *Commentary* volume. Reports identifying the major differences between the 1997 and the 2000 Editions of the *Provisions* and describing unresolved issues and major technical topics in need of further study are also being prepared.

Code-language versions of changes for the 2000 *Provisions* for submittal as proposed code changes for the 2003 Edition of the *IBC* are being developed for the BSSC by S. K. Ghosh Associates. In addition, the *Provisions* are undergoing a detailed edit to eliminate undue repetition and inconsistencies; this document is expected to serve as the base document for the 2003 update cycle. Prior to submittal to the BSSC member organizations for comment/ballot, the PUC and Board will review the *IBC* change proposals and edited version of the *Provisions* in late spring 2001.

Planning for 2003 Update

As part of the preliminary planning effort, NIBS contracted with FEMA for a study to permit BSSC to explore how best to make use of new technology (e.g., the Internet for balloting) in the 2003 *Provisions* update cycle and beyond. An additional task involved the convening of an expert group to formulate how best to deal with nonbuilding structures in the update process. This meeting was held in January 2001, and a draft report has been developed for consideration by the PUC and BSSC Board.

FEMA has developed a paper expressing its view of the current situation with respect to model codes and standards and the *Provisions* role in this process. This paper was presented to the PUC and BSSC Board at meetings on March 5 and 6, 2001, and to the BSSC Annual Meeting on March 7.

Code Resource Development Effort

In mid-1996, FEMA asked the BSSC to initiate an effort to generate a code resource document based on the 1997 *Provisions* for use by the International Code Council (ICC) in adopting seismic provisions for the first edition of the *International Building Code (IBC)* to be published in 2000. The Code Resource Development Committee (CRDC) appointed to conduct this effort met several times over the next year to develop a code language/format version of the 1997 *Provisions*, and the CRDC-developed draft requirements were presented to the *IBC* subcommittee by Gerald Jones in March 1997.

Subsequently, the CRDC met to develop comments on the *IBC* working draft to be submitted to the ICC in preparation for an August 1997 public comment forum. These comments generally reflected actions taken by the PUC in response to comments submitted with the first ballot on the changes proposed for the 1997 *Provisions* as well as CRDC recommendations concerning changes made by the *IBC* Structural Subcommittee in the original CRDC submittal. CRDC representatives attended the August forum to support the CRDC recommendations.

After issuance of the first draft of the *IBC* in November 1997, the CRDC met to prepare "code change proposals" that reflected the final version of the 1997 *Provisions* for submittal in January 1998. The CRDC then met for the last time as a committee in March 1998 to review the compilation of *IBC* code change proposals issued by the ICC and to develop a strategy for supporting the code change proposals it had developed at an *IBC* public hearing in April. In addition, the *IBC* Structural Subcommittee asked for CRDC input concerning all the seismic-related code change proposals and these comments were summarized and transmitted to the *IBC* group for its consideration.

An eight-member Code Resource Support Committee (CRSC) then was established to support the *Provisions*-based requirements through the remainder of the adoption process and to provide for needed liaison with the 2000 *Provisions* development work. A CRSC Technical Group composed of representatives of the 2000 PUC and the various materials interests also was established to support the CRSC. The first task of the CRSC was to deal with one major issue that arose at the April hearing at which several code change proposals concerning the draft *IBC* (and 1997 *Provisions* based) response modification factors and limits of applicability of certain structural systems were discussed. At the suggestion of a CRDC representative at the hearing, the proponents of those code changes agreed to withdraw their proposals to permit discussion of their technical merit outside the forum of the public hearing process. To this end, the CRSC invited these code change proponents as well as representatives of the various construction industry materials associations to an August 1998 meeting at which the group formulated a consensus opinion on an appropriate series of code change proposals that could be submitted to replace those withdrawn in April. Additional topics also were discussed and a total of 13 code-change proposals were drafted.

In September 1998, the 2000 PUC Executive Committee was briefed on these code-change proposals, most of which were accepted by the PUC as items to be considered during the 2000 update effort; however, five items were deemed to be significant departures from the 1997 *Provisions* and required a vote by the full PUC. This balloting concluded in early October with all items achieving consensus approval. The CRSC then finalized all 13 of its code change proposals and submitted them to the ICC in late October 1998.

In January and February 1999, the CRSC met with its Technical Group to consider the proposed changes to the *International Building Code* seismic provisions that would be debated at March 1999 hearings. The CRSC chair and several member participated in the hearings on behalf of the CRSC.

An *International Residential Code* Task Group established within the CRDC in late-1997 has provided the ICC committee developing the *International Residential Dwelling Code* (*IRC*) with input concerning seismic requirements reflecting the 1997 *Provisions*, and these requirements generally were reflected in the draft *IRC*. The activities of this task group have parallel those of the CRDC/CRSC with the *IBC* and representatives attended the *IRC* July 1998 public hearing in Kansas City. At this hearing, agreement was reached on the seismic map to be included in the *IRC*; this map subsequently was prepared for the BSSC by USGS and submitted to the ICC for inclusion in the final draft of the *IRC*. The task group met in February 1999 to review proposed code changes and prepare for the March ICC hearings.

The CRSC chair and several CRSC members represented the group at the joint annual conference of BOCA, ICBO, and SBCCI held in September 1999 in St. Louis. Overall, the CRSC was successful in that almost all challenges to the seismic provisions were decided in favor of the CRSC position and the seismic provisions in both the 2000 *International Building Code* and the *International Residential Code* reflect the 1997 *NEHRP Recommended Provisions*.

In preparation for the ICC hearings to be held in Birmingham, Alabama, in April 2000, the CRSC and its Technical Group reviewed the code changes and met via telephone conference calls in March 2000 to discuss the proposals. The CRSC chair and several other CRSC members attended the hearings. With respect to the *International Building Code*, the CRSC had specific positions on 41 proposals. Of these proposals, 35 were decided in the direction CRSC favored and two that the CRSC opposed were withdrawn. During the hearings on the *International Residential Code*, the CRSC had specific positions on 12 proposals. Eight of these proposals were decided in favor of CRSC's position and one was withdrawn at CRSC's request.

In late September 2000, NIBS entered into a contract with FEMA to fund further code support work by the BSSC. Under this contract, the BSSC is charged to: (a) update and expand the Code Resource Support Committee (CRSC) to ensure that it continues to be staffed with appropriate experts from the seismic design and code development communities; (b) convert the changes in the 2000 edition of the *NEHRP Recommended Provisions* into code language; (c) submit those *Provisions* changes as code changes for the 2003 *IBC* and *IRC* code change cycle; (d) continue to monitor the *IBC* and *IRC* code change process to ensure that code changes submitted by other parties do not reduce the effectiveness of the *IBC* and *IRC* seismic provisions to the end that the codes would no longer be substantially equivalent to the *NEHRP Recommended Provisions*; (e) provide an equal level support to the recently announced building code development process being undertaken by National Fire Protection Association (NFPA); and (f) encourage the use of adequate building codes at the local level as part of FEMA's commitment to predisaster mitigation, especially for communities participating in FEMA's Project Impact initiative, including assisting local community code officials in adopting and enforcing a suitable code and assisting in the interpretation of that code as it is used by the local community.

The 2001 CRSC now has been reconstituted to include additional members and two special task groups; one to focus on the *IRC*, and one to focus on the NFPA code. The expanded CRSC and its Technical Advisory Group (TAG) has reviewed the proposals for change to the *IBC* and *IRC* in preparation for the hearing to be held in Portland, Oregon, in late March. During a February 23 conference call, the CRSC formulated its position on the proposed changes to the *IRC*. At a meeting on March 5, with its TAG, the CRSC decided upon its positions on the proposed changes to the *IBC*. The CRSC chair and six members are expected to attend the Portland hearings.

The CRSC's NFPA Task Group members also have been attending meetings of the NFPA Technical Correlating Committee (TCC) and Structures and Construction Committee. In addition, the CRSC representative to the TCC has been appointed by that committee as its representative to the Performance Task Group to the Fundamentals Committee.

Information Dissemination/Technology Transfer

The BSSC continues in its efforts to stimulate widespread use of the *Provisions*. In addition to the issuance of a variety of publications that complement the *Provisions*, over the past seven years the BSSC has developed materials for use in and promoted the conduct of a series of seminars on application of the *Provisions* among relevant professional associations. To date, more than 90 of these seminars have been conducted with a wide variety of cosponsors and more than 75,000 reports have been distributed.

Other information dissemination efforts have involved the participation of BSSC representatives in a wide variety of meetings and conferences, BSSC participation in development of curriculum for a FEMA Emergency Management Institute course on the *Provisions* for structural engineers and other design professionals, issuance of press releases, development of in-depth articles for the publications of relevant groups, work with Building Officials and Code Administrators International (BOCA) that resulted in use of the *Provisions* in the BOCA *National Building Code* and the Southern Building Code Congress International's *Standard Building Code*, and cooperation with the American Society of Civil Engineers (ASCE) that resulted in use of the *Provisions* in the 1993 and 1995 Editions of Standard ASCE 7. In addition, many requests for specific types of information and other forms of technical support are received and responded to monthly.

During 1996, as part of the efforts of a joint committee of the BSSC, Central U.S. Earthquake Consortium, Southern Building Code Congress International and Insurance Institute for Property Loss Reduction to develop mechanisms for the seismic training of building code officials, the BSSC contributed its expertise in the development of a manual for use in such training efforts.

Information dissemination efforts on the 1997 edition of the *Provisions* were somewhat curtailed pending incorporation of those requirements into the International codes. Work was initiated on developing a new edition of *Nontechnical Explanation of the NEHRP Recommended Provisions* to reflect the 1997 *Provisions* and a new edition of *Seismic Considerations for Communities at Risk* that reflects the 1997 *Provisions* as well as the new *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, the new HAZUS loss estimation methodology, and FEMA's Project Impact. Further, some of the instructional course materials developed to reflect the 1991 and 1994 editions of the *Provisions* have been updated to reflect changes made for the 1997 edition.

In September 1999, NIBS entered into a 60-month indefinite quantity contract with FEMA for conduct of the BSSC's information dissemination. The first task order issued under the contract charges the BSSC to increase its capability to respond to requests for technical assistance relating to the *Provisions*, to increase its capability to provide more general technical assistance and information in a coordinated and proactive manner and using all communication media including an Internet web site currently being developed, to revise the course materials including the *Guide to Application of the Provisions*, an Instructors Manual and slide set, and a Student Manual to reflect the 2000 *Provisions* and the code requirements based on the *Provisions*, to prepare and implement a plan to market the instructional materials and subsequently conduct an ongoing series of instructional (both technical and nontechnical) training seminars on an as-requested basis, to continue to promote and encourage the use of the *Provisions* by the nation's model code organizations and their adoption by local jurisdictions, and to continue to conduct activities to increase the general awareness of the earthquake risks in different regions throughout the country and the need to use local building codes that are substantially equivalent with the *Provisions*.

IMPROVING THE SEISMIC SAFETY OF EXISTING BUILDINGS

Guidelines/Commentary Development Project

The 1997 *NEHRP Guidelines for the Seismic Rehabilitation* and *Commentary* volumes and 1997 map packet (which also include maps referenced in the *NEHRP Recommended Provisions for New Buildings and Other Structures*) are readily available as are two companion volumes – *Planning for Seismic Rehabilitation: Societal Issues* (FEMA 275) and *Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 276).

Case Studies Project

The case studies project was an extension of the multi-year project leading to publication of the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* and its *Commentary* in late 1997. The project is expected to contribute to the credibility of the *Guidelines* by providing potential users with representative real-world application data and to provide FEMA with the information needed to determine whether and when to update the *Guidelines*. The final report on the project was delivered to FEMA in September 1999 and is now available as FEMA 343, *Case Studies: An Assessment of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings.*

Guidelines Training Seminars

In August 1997, NIBS entered into a contract with FEMA for the design and conduct of a series of technical training seminars to transfer the technology and information contained in the *Guidelines* to structural and architectural engineers (whether in private or government practice, representing organizations both large and small); to local building officials and technical staffs, interested contractors, and mitigation officials, where applicable; and to engineering educators and students in institutions offering seismic design curricula. Conceptually, the seminar curriculum will take the form of a series of modules that will permit it to be adapted for use with a variety of audiences.

The Applied Technology Council, under contract to the BSSC, developed the seminar program syllabus and other instructional materials. To date, approximately 2000 structural engineers have attended seminars on the *NEHRP Guidelines for \the Seismic Rehabilitation of Buildings*. Being conducted for FEMA by the BSSC with the assistance of the Applied Technology Council, two-day seminars have been held in San Diego; Salt Lake City; Portland, Oregon; Los Angeles; Seattle; New York City; Oakland; St. Louis; Charleston, South Carolina; Chicago, Illinois; Sacramento, California; and Washington, D.C.

BSSC MEMBER ORGANIZATIONS
(* indicates affiliate nonvoting member)

AFL-CIO Building and Construction Trades Department
AISC Marketing, Inc.
American Concrete Institute
American Consulting Engineers Council
American Forest and Paper Association
American Institute of Architects
American Institute of Steel Construction
American Insurance Services Group, Inc.
American Iron and Steel Institute
APA - The Engineered Wood Association
American Society of Civil Engineers
American Society of Civil Engineers--Kansas City Chapter
American Society of Heating, Refrigeration, and
Air-Conditioning Engineers
American Society of Mechanical Engineers
American Welding Society
Applied Technology Council
Associated General Contractors of America
Association of Engineering Geologists
Association of Major City Building Officials
Bay Area Structural, Inc.*
Brick Industries Association
Building Officials and Code Administrators International
Building Owners and Managers Association International
Building Technology, Incorporated*
California Geotechnical Engineers Association
California Division of the State Architect, Office of Regulation Services
Canadian National Committee on Earthquake Engineering
Concrete Masonry Association of California and Nevada
Concrete Reinforcing Steel Institute
Eagle Point Software*
Earthquake Engineering Research Institute
General Reinsurance Corporation*
General Services Administration Seismic Program
Hawaii State Earthquake Advisory Board
HLM Design*
Institute for Business and Home Safety
Interagency Committee on Seismic Safety in Construction
International Conference of Building Officials
International Masonry Institute
Masonry Institute of America
Metal Building Manufacturers Association
Mid-America Earthquake Center
National Association of Home Builders
National Concrete Masonry Association
National Conference of States on Building Codes and Standards
National Council of Structural Engineers Associations
National Elevator Industry, Inc.
National Fire Sprinkler Association
National Institute of Building Sciences
National Ready Mixed Concrete Association
Permanent Commission for Structural Safety of Buildings*
Portland Cement Association
Precast/Prestressed Concrete Institute
Rack Manufacturers Institute
Seismic Safety Commission (California)
Southern Building Code Congress International
Southern California Gas Company*
Steel Deck Institute, Inc.
Steel Joist Institute*
Steven Winter Associates, Inc.*
Structural Engineers Association of Arizona
Structural Engineers Association of California
Structural Engineers Association of Central California
Structural Engineers Association of Colorado
Structural Engineers Association of Illinois
Structural Engineers Association of Northern California
Structural Engineers Association of Oregon
Structural Engineers Association of San Diego
Structural Engineers Association of Southern California
Structural Engineers Association of Utah
Structural Engineers Association of Washington
The Masonry Society
U. S. Postal Service*
Western States Clay Products Association
Western States Council Structural Engineers Association
Westinghouse Electric Corporation*
Wire Reinforcement Institute, Inc.

BUILDING SEISMIC SAFETY COUNCIL PUBLICATIONS

Available free from the Federal Emergency Management Agency at 1-800-480-2520 (order by FEMA Publication Number). For detailed information about the BSSC and its projects, contact: BSSC, 1090 Vermont Avenue, N.W., Suite 700, Washington, D.C. 20005 Phone 202-289-7800; Fax 202-289-1092; e-mail ctanner@nibs.org

NEW BUILDINGS PUBLICATIONS

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 2000 Edition, 2 volumes and maps, FEMA 368 and 369

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 1997 Edition, 2 volumes and maps, FEMA 302 and 303

Guide to Application of the 1991 Edition of the NEHRP Recommended Provisions in Earthquake Resistant Building Design, Revised Edition, 1995, FEMA 140 – new edition in preparation

A Nontechnical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1995, FEMA 99 – new edition expected to be published in late 1999 or early 2000

Seismic Considerations for Communities at Risk, Revised Edition, 1995, FEMA 83 – new edition expected to be published in late 1999 or early 2000

Seismic Considerations: Apartment Buildings, Revised Edition, 1996, FEMA 152

Seismic Considerations: Elementary and Secondary Schools, Revised Edition, 1990, FEMA 149

Seismic Considerations: Health Care Facilities, Revised Edition, 1990, FEMA 150

Seismic Considerations: Hotels and Motels, Revised Edition, 1990, FEMA 151

Seismic Considerations: Office Buildings, Revised Edition, 1996, FEMA 153

Societal Implications: Selected Readings, 1985, FEMA 84

EXISTING BUILDINGS

NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1997, FEMA 273

NEHRP Guidelines for the Seismic Rehabilitation of Buildings: Commentary, 1997, FEMA 274

Case Studies: An Assessment of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1999, FEMA 343

Planning for Seismic Rehabilitation: Societal Issues, 1998, FEMA 275

Example Applications of the NEHRP Guidelines for the Seismic Rehabilitation of Buildings, 1999, FEMA 276

NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings, 1992, FEMA 172

NEHRP Handbook for the Seismic Evaluation of Existing Buildings, 1992, FEMA 178

An Action Plan for Reducing Earthquake Hazards of Existing Buildings, 1985, FEMA 90

MULTIHAZARD

An Integrated Approach to Natural Hazard Risk Mitigation, 1995, FEMA 261/2-95

LIFELINES

Abatement of Seismic Hazards to Lifelines: An Action Plan, 1987, FEMA 142

Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of An Action Plan, 6 volumes:

Papers on Water and Sewer Lifelines, 1987, FEMA 135

Papers on Transportation Lifelines, 1987, FEMA 136

Papers on Communication Lifelines, 1987, FEMA 137

Papers on Power Lifelines, 1987, FEMA 138

Papers on Gas and Liquid Fuel Lifelines, 1987, FEMA 139

Papers on Political, Economic, Social, Legal, and Regulatory Issues and General Workshop Presentations, 1987, FEMA 143

(March 2001)